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FLOOD MANAGEMENT STUDY OF THE C-18 BASIN

by
Steve S. T. Lin

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Water Resources Division
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EXECUTIVE SUMMARY

Increasing population and rapid land development in the coastal floodplain of the C-18 basin have necessitated updated flood management studies to insure adequate flood protection without impacting the environment and water resources. Currently the C-18 basin is 70 percent under the wetland category with urban use representing 10 percent of the basin and the remainder in agricultural use. A substantial area is currently proposed to be developed for residential and industrial use. For example, MacArthur Foundation was granted a surface water management conceptual approval for 16,575 acres within the C-18 basin, of which approximately 5,300 acres will be preserved as water management areas.

The purpose and scope of this study is to determine the flood characteristics of the C-18 basin under present land use conditions, and provide the discharge limitations and degree of flood protection of the existing system. This information will provide a basis for future stormwater management criteria in the basin.

This report presents the methodology, findings, and recommendations for flood management and runoff limitations of the C-18 project and its sub-basins. The results of this study can be summarized briefly as follows:

1. Water control structure S-46 is capable of handling the 100 year design discharge. The east branch and main canal of C-18 can handle 25-30 year design flows. The western half of the west branch and the reach upstream of the C-18 weir lack the capacity to pass the 10-year[1] discharge frequency used in this study.

2. The computed discharge at S-46 for 10-, 25-, and 100-year design storms are 2050 cfs, 2280 cfs, and 2714 cfs, respectively.

3. Present discharge rates from several C-18 subbasins are far below their permitted allocations due to high tailwater conditions resulting from limited channel capacity. For those permitted existing developed areas, the permitted house pad elevations are above the computed flood stages, because the permit decisions accounted for limited outfall contingencies.

4. Runoff from several subbasins lacks a positive outfall. Current model results indicate C-18 is not capable of transporting any additional inflow from these areas. Future discharge from these subbasins should be reevaluated considering future scenarios for basin water management and environmental protection.

5. The average runoff rate from subbasins upstream of the C-18 weir is 0.37 inch per day. This is far less than the rate of 1 inch per day allowed under previous permit criteria. Consequently, these areas will require substantial stormwater detention and tailwater consideration for flood protection. Compensating storage should be provided consistent with District's regulatory criteria. Outflow rates to the C-18 canal should be maintained at the present level.

6 Design flood profiles and stages for C-18 are presented for use in storm water management system design and compliance with regulatory criteria such as compensating storage and design storm routings.

7. New discharge criteria are recommended to be applied to new permit applications and in the modification of existing permits.

[1]10-year discharge refers to the discharge which has a one in ten year return period. This nomenclature is utilized throughout the report for storms (rainfall), discharges, floods, and stages.

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ACKNOWLEDGMENTS

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ABSTRACT

The C-18 basin is located in the northeastern portion of Palm Beach County and consists of about 100 square miles of land tributary to the Southwest Fork of the Loxahatchee River. Increasing population and rapid land development in the area has necessitated updated flood management studies to insure adequate flood protection.

This basin is part of the coastal floodplain of the lower east coast of south Florida, and is characterized by extremely flat terrain and heavy vegetation interspersed with numerous wetlands. In performing the flood management studies, somewhat unique concerns in south Florida are the backwater conditions due to tidal and nontidal tailwater, flow reversals from project canals to low flat lands, and flow diversion by weir, culvert, and pumping facilities. The EPA EXTRAN model, a dynamic wave routing model, was used as a method of analysis. The use of storage options which provide a direct hydraulic link between a subbasin and the project canal minimizes the difficulties in modeling the unique hydrologic characteristics of south Florida. The model was calibrated with the severe storm event in the basin of September 21-24, 1983. The design storms of 10, 25, and 100 year events were evaluated based on present land uses and water management in the C-18 basin. The discharge limitations and the degree of flood protection of the existing system are defined in the present study. Future stormwater management permiting criteria are recommended.

I. INTRODUCTION AND STUDY SCOPE

The C-18 basin has been faced with numerous water-related problems. Increasing land development in the area is placing greater demands on the area's resources. Rapid conversion of former open pine flatwoods and wetlands to residential and citrus grove development has renewed concern over the area's water resource needs for flood protection, water quality, water supply, and environmental preservation and enhancement.

Several reports and studies have been done in this basin by the U. S. Army Corps of Engineers (1956, 1980), the U. S. Geological Survey (1972, 1973, 1980), the U. S. Fish and Wildlife Service, and the Florida Game and Freshwater Fish Commission (1981). The U. S. Department of the Interior National Park Service published the Loxahatchee River Wild and Scenic River Study and Environmental Impact Statement in 1982. The latest study was the Feasibility Report and Environmental Assessment for Canal 18-Loxahatchee Slough by the U. S. Army Corps of Engineers (Corps), dated June 1983. In this report, flooding conditions under 1979 land use for 10-, 25-, 50-, and 100-year[1] and standard project storms were analyzed. The DAM-BREAK version of the Hydrologic Engineering Center, "Flood Hydrograph Package" (HEC-1) was applied. However, the following features that exist in the C-18 basin were not considered.

- The HEC-1 computer runs did not simulate the discharge characteristics of culverts with stop log risers under varying headwater and tailwater conditions, nor did they simulate the time varying stage discharge or tailwater effects of both the channel and culvert flow or the operating schedule of control structure S-46.
- The unit hydrograph characteristics employed in the design of the C-18 project may have been inappropriate. The time of concentration and 6-hour unit hydrograph used in subbasin hydrograph development may have been excessive.
- Design rainfall was based on three long-term raingage stations (Loxahatchee Grove, St. Lucie New Lock 1, and West Palm Beach Airport), all of which are outside the C-18 basin.
- There has been a change in gate operation at S-46 since the Corps study which has reduced the outflow peak rate and extended the storm runoff duration.

The purpose and scope of the present study is to determine flood characteristics of the C-18 basin under present land use conditions by using a different approach than the one used by the Corps. This report presents the methodology, findings of the study, and recommendations relating to stormwater management and permitting criteria.

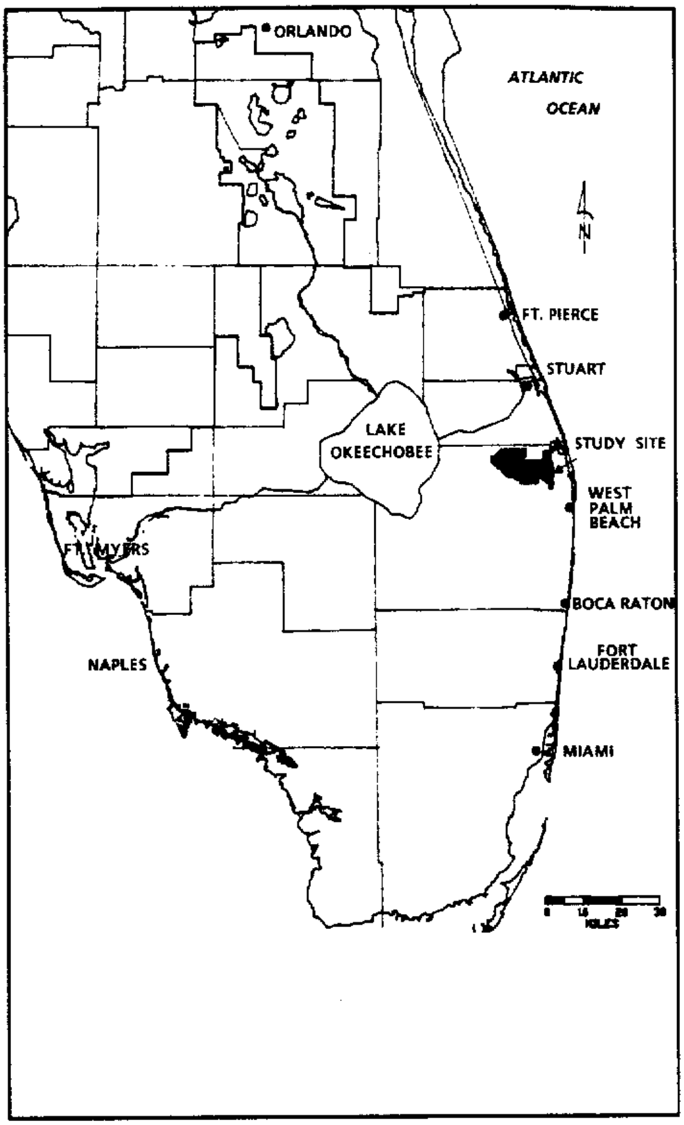

FIGURE 1. Location Map of C-18 Study Area

II. STUDY AREA DESCRIPTION

A. Location

The C-18 basin is located in the northeasterly portion of Palm Beach County and includes the northerly portion of the Loxahatchee Slough and the lands west of the slough to the low divide between the basin and the L-8 canal drainage area (Figures 1 and 2). The watershed is bounded on the south by the C-17

[1]10-year discharge refers to the discharge which has a one in ten year return period. This nomenclature is utilized throughout the report for storms (rainfall), discharges, floods, and stages.

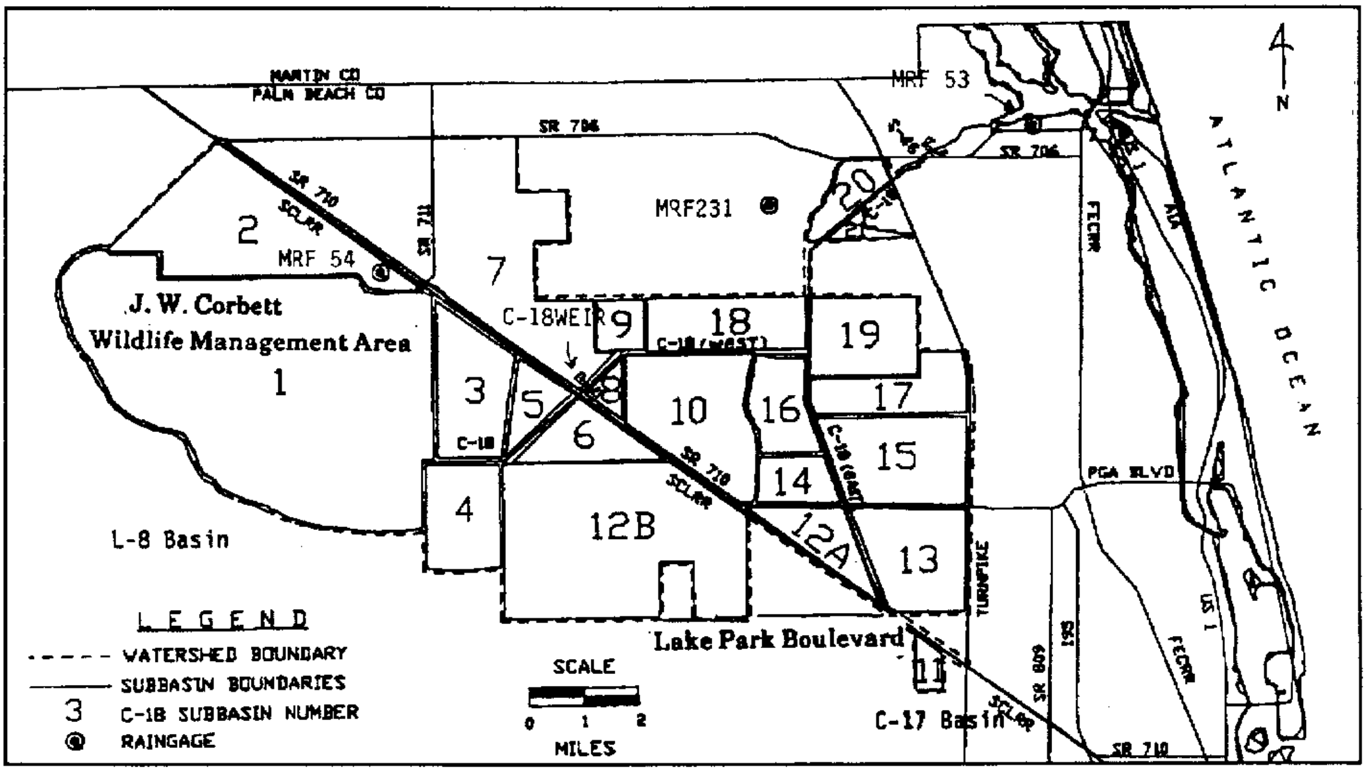

FIGURE 2. Vicinity Map of C-18 Basin

drainage area (Lake Park Boulevard), and on the east by the Florida Turnpike. The drainage area is approximately 100 square miles, of which about 21 square miles on the western side are within the J. W. Corbett Wildlife Management Area.

B. Hydrology

1. Rainfall and Runoff. The climate of the study area is subtropical with the average daily temperature ranging from 68°F in the winter to 82°F in the summer. Rainfall occurs in fairly distinct cycles, with the wet season extending from May to October. There are three raingage stations within the C-18 basin. They are South Indian River Water Control District (SIRWCD), Jupiter Fire Station, and Pratt & Whitney. The maximum and minimum annual rainfall at Pratt & Whitney was 98.29 inches (1983), and 42.45 inches (1961) with an overall average of about 61 inches per year. The maximum recorded daily rainfall at SIRWCD was 9.5 inches on September 24, 1983 (Table B-1, Appendix B).

Prior to the construction of C-18, the area was typically drained by sheet flow to the north. Drainage patterns were not well defined. During periods of high rainfall, many low-lying areas and wetland communities were inundated with water. During drought periods the water table was within 1 foot of the surface. After the C-18 project was completed in 1958, runoff was diverted to tidewaters through S-46 at the Southwest Fork of the Loxahatchee River.

Table 1 presents annual peak daily discharge at S-46 and maximum 3-day rainfall at rainfall stations SIRWCD (MRF-231, 1978-Present), Jupiter Fire Station (MRF-53, 1960-1975), and Pratt & Whitney (MRF-54, 1957-Present). Rainfall distribution over the basin is fairly uniform under high rainfall conditions. The peak daily discharge is not directly proportional to the rainfall amount. This may be due to time lags, time of year, antecedent condition and changed water management practices in the basin.

The general land elevation in the C-18 basin ranges from 14 feet NGVD in the Loxahatchee Slough to 25 feet NGVD in the northwest portion of the basin. Numerous wetland depressions are scattered throughout the basin. The eastern divide is relatively low, about 1 to 2 feet higher than the general ground elevations within the Loxahatchee Slough (approximately 17.0 feet NGVD). The terrain to the west in the J. W. Corbett Wildlife Management Area is also flat. Because of the flat topography, the drainage is poor; therefore, following periods of heavy rainfall much of the land is inundated.

TABLE 1. ANNUAL PEAK DAILY DISCHARGE AT S-46 WITH CORRESPONDING 3-DAY RAINFALL IN THE BASIN FOR PERIOD OF RECORD (1959-1985).

Year	Annual Peak Daily Discharge at S-46 cfs	Maximum 3-Day Rainfall-Inches at Stations MRF-231	MRF-53	MRF-54
1959	2330			10.17
1960	2730		9.00	7.60
1961	266		4.89	4.95
1962	1040		3.55*	4.10
1963	1160		9.56	3.70
1964	1200		5.38	8.85
1865	1140		4.50	6.00
1966	2216		8.20	5.85
1967	971		4.43	4.30
1968	543		7.23	4.95*
1969	425		4.66	5.60
1970	1960		8.69	9.05
1971	1290		3.95*	4.85
1972	1340		8.01	2.70*
1973	1030		5.42	4.70
1974	531			7.65
1975	359			4.30
1976	660			7.60
1977	471			5.65
1978	1310			11.90
1979	331	3.14		8.65
1980	262	3.25*		3.20
1981	772	7.93		6.55
1982	1859	7.55		7.80
1983	1972	11.29		12.55
1984	2178	9.81		12.10
1985	492	5.39		5.70

* = same event but not the maximum rainfall in the year

The Loxahatchee Slough is located in the eastern half and was originally about 3 miles wide east to west, and about 15 miles long north to south. The slough area consists of flat terrain with an average elevation of about 17.0 feet NGVD. It is interspersed with numerous ponds and is subject to long periods of inundation during the wet season. The general topography of the C-18 basin is presented in Figure 3.

2. Land Use. During 1956 when the Corps was preparing the General and Detail Design Memorandum for C-18 and water control structure S-46, there were about 4,300 acres of improved and semi-improved beef pasture land within the C-18 basin. The Corps study predicted there would be no urban or suburban development in the C-18 basin during the next 50 years, with or without the C-18 project. The present land use, however, is quite different from the one predicted by the Corps. Based on field investigation and permit information as of 1985,there are now slightly over 10,000 acres of agricultural land and about 6,900 acres of urban development, including industrial parks. Table 2 presents existing land uses in each subbasin of the C-18 basin. Note that about 70% of the basin is under the wetland category.

3. Soil Associations. The following general soil associations are found in the C-18 basin (Figure 4):

a. Wabasso-Riviera Association: This association consists of nearly level, poorly drained, sandy soils that have a loamy subsoil; some have a weak cemented sandy layer over the loamy subsoil. This association is in the area of Pratt & Whitney, Caloosa Estates, and along the area north and south of the west branch of C-18. The minor soils in this association are Pineda and Oldsmar soils. A high water table is a severe limitation for most farm uses. With adequate water control, these soils are well or moderately well suited to citrus, truck crops, and improved pasture. Drainage and fill are required to make some areas suitable for building sites.

b. Riviera-Boca Association: This association is nearly level, poorly drained, sandy soils that have a loamy subsoil; some are moderately deep over limestone. This association is mostly in the J. W. Corbett Wildlife Management Area. The minor soils in this association are Floridana and Hallandale soils. Much of this association is in native vegetation, and is severely limited for most farm uses due to a high water table.

c. Winder-Tequesta Association: This association is a nearly level, poorly drained sandy

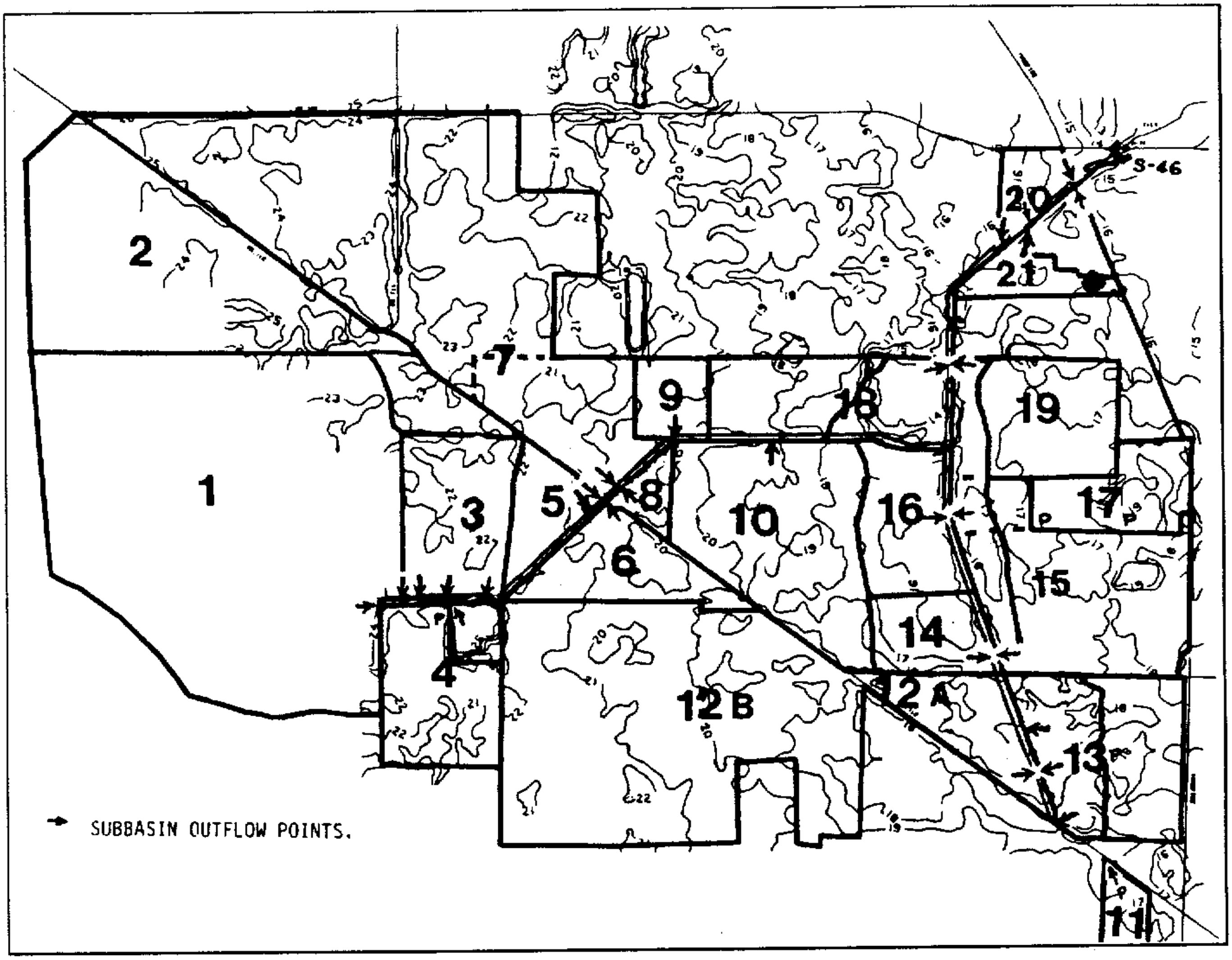

FIGURE 3. Topographic and Subbasin Outflow Points in the C-18 Basin

soil that has a loamy subsoil, and possibly a thin layer of muck at the surface. This association is in the Loxahatchee slough area along both sides of the east branch of C-18 and in a portion of the main canal. Winder soils have a thin surface layer of black fine sand and a subsurface layer of light gray and brownish gray fine sand. Below this is a thin layer of loamy fine sand that rests on gray fine sand mixed with white shell fragments. Tequesta soils have approximately a 12-inch layer of black muck at the surface. The subsoil is grayish brown fine sandy loam. The minor soils in this association are Riviera and Pahokee soils. Pahokee soils have a surface layer of black muck. Below this is black and dark reddish brown muck that rests on hard limestone at a depth of approximately 42 inches. This formed the predevelopment, natural drainage way, and the soils are subject to flooding for long periods.

d. Holopaw and Myakka Series: Holopaw fine sand is a nearly level, poorly drained soil that has a thick sandy surface layer and a loamy subsoil at a depth of 40 to 72 inches. Under natural conditions, the water table is within 10 inches of the surface for 2 to 6 months during most years. If drained and intensively managed, it is moder-ately suited to vegetables. This soil covers a minor portion of the C-18 basin.

e. Myakka Sand: this soil also covers a minor portion of the C-18 basin, and is a nearly level, poorly drained, deep sandy soil which has a dark colored layer, weakly cemented with organic matter. Under natural conditions, the water table is within 10 inches of the ground surface for 2 to 4 months in most years. This soil is moderately well suited to vegetables if irrigation water is available.

TABLE 2. PRESENT LAND USE IN THE C-18 BASIN. DATA BASED ON 1985 SURVEY (UNITS IN ACRES)

Subbasin No.	Agricultural	Urban	Wetland	Lake/Water	Forest	*Remarks
1			13387			
2	15	849*	6690	101		Industrial
3	1748*		171			Open & to be developed
4	1591*	28	281			Citrus
5	8	25	780	7		
6	405		729	3		
7		1643	113			
7A	171	148	7186	10	163	
8	6	255		12		
9		598*	6			Estate
10	2427		795	65		
11	350					
12			989	33	185	
12A	2196	10	5246	8	128	
13	81	1708	82	154	80	
13A		47	245	30		
14			853		100	
15	267	803	1406	22	249	
15A			366	33		
16	2		1291	21	164	
17	447	663		26	3	
18			1152			
18A		13	882	17		
19	9	3	1473			
19A			596	53	6	
20	436	1	60		10	
21		100	186	27		
TOTAL	10159	6894	44965	622	1088	63728
%	15.94	10.82	70.56	0.98	1.70	100

C. Water Management

Prior to the C-18 project, the original canal (called Limestone Creek) had an irregular hydraulic prism which varied from a 20-foot bottom width at an elevation of about -5.0 feet NGVD near the Southwest Fork of the Loxahatchee River to a narrow ditch with a bottom elevation at 12.0 feet NGVD about 3.5 miles southwest of State Road 706/C-18 crossing. This original canal was extended and widened during the late 1950s with a 6.1 mile canal in a south-north direction (called east branch of C-18), and 7.9 miles of the west branch of C-18 to drain the western portion of the basin. A control structure, S-46, was constructed near the outlet of C-18 to regulate water levels and prevent salt water intrusion. Figure 5 presents comparisons of existing C-18 bottom profiles and design grades. There are several humps in the canal bottom, and the depth of water in C-18 is generally shallow (3 to 10 ft in depth).

Water management in the C-18 basin is largely regulated through the operation of S-46. A number of local private landowners and subdivisions operate small pumps and water control structures as part of the secondary drainage systems under SFWMD permits. The subbasin outflow points are presented in Figure 3.

Water control structure S-46 is a reinforced concrete, gated spillway with discharge controlled by three stem automatically operated, vertical lift gates. This structure is located in C-18 approximately 2,400 feet east of the Florida Turnpike, maintains an optimum upstream stage of 14.8 feet NGVD, and is designed to pass the design flood of 3,420 cfs (50% of the standard project flood). The automatic controls on the gates function as follows:

- When the headwater stage rises to 15.0 feet NGVD, the gates open at a rate of 0.4 inches per minute. Due to complaints of erosion downstream of S-46, the speed of the gates' movement has been modified from an opening rate of 6 inches per minute prior to December 1982, to the present rate of 0.4 inches per minute.
- When the headwater stage rises or falls to 14.8 feet NGVD, the gates become stationary.
- When the headwater stage falls to 14.5 feet NGVD, the gates close at the rate of 0.4 inch per minute.

The C-18 Weir, a steel sheet pile weir, located about 200 feet downstream of the Beeline Highway (SR710), maintains an optimum upstream stage of 17.6 feet NGVD to prevent overdrainage. The design capacity of this weir is 190 cfs (30% standard project

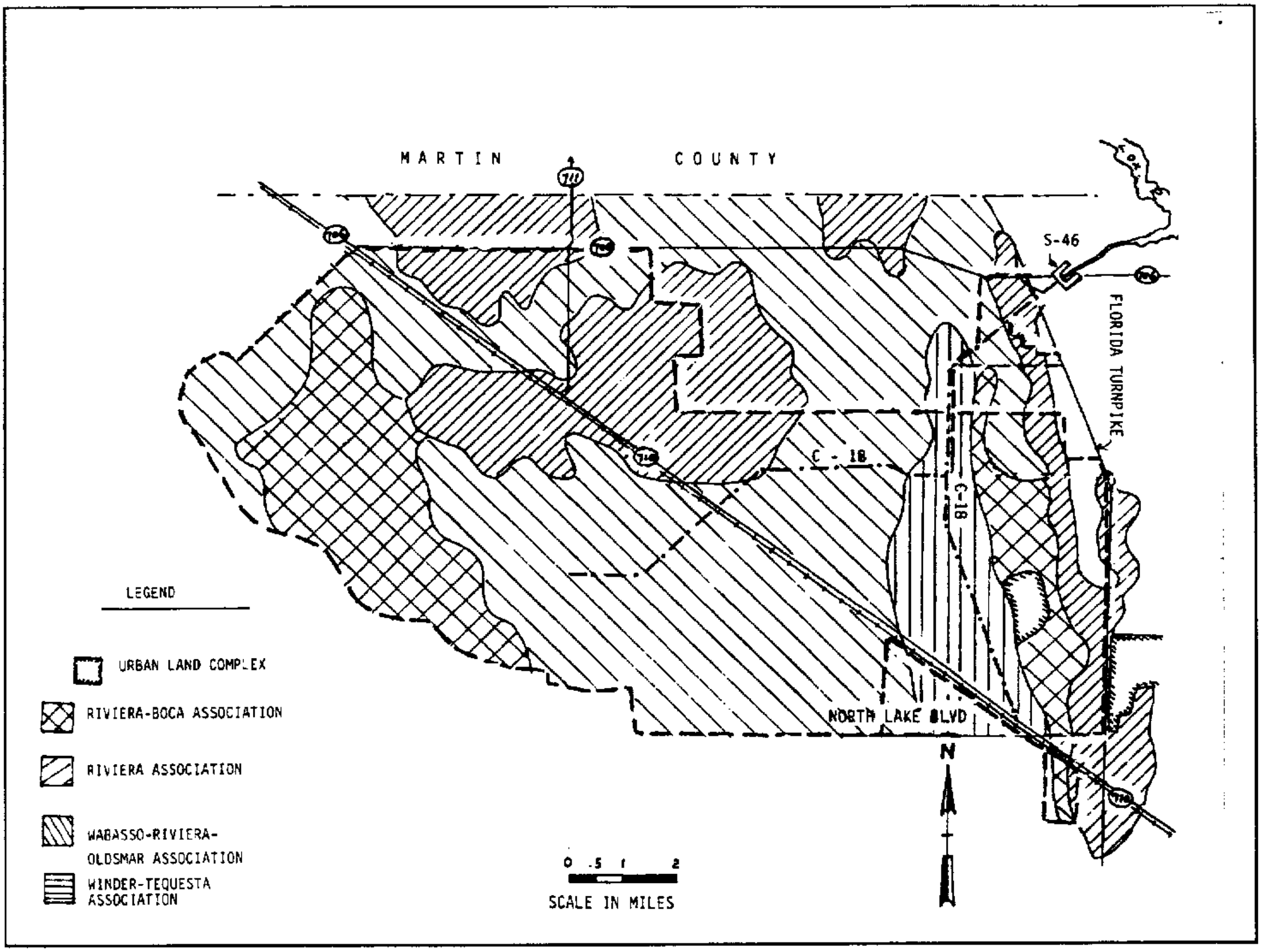

FIGURE 4. General Soil Map in the C-18 Basin

flood). This 190 cfs approximates the existing channel capacity.

During major storm events the gates at S-46 are operated manually to lower and maintain a headwater stage of 12.8 ft NGVD. A major storm event is defined as any event which causes a tailwater stage at the C-18 weir to rise above 17.6 ft. Once the tailwater stage at the weir falls below 17.6 ft NGVD, the headwater stage at S-46 is allowed to rise gradually.

Present surface water runoff allocation for the C-18 basin is based on the following criteria: For the area upstream of the C-18 weir, and the area south of State Road 710, the permissible discharge rate is 1 inch per 24 hours under a 25 year design storm. For the area east of State Road 710, the allowable discharge rate was based on the equation $Q = (114/\sqrt{A}+34)*A$, where Q is the peak discharge in cfs and A is the drainage area in square miles. This equation is called the "Old Everglades Runoff Formula," and was developed by the Everglades Drainage District. Two fixed points used to define the formula's coefficients are:

1. The estimated runoff from a one square mile tract under 1 in 25 year event was equivalent to 148 cfs. This event represents a rainfall amount of 11 inches in 24 hours with rainfall excess (direct runoff) of 5.5 inches.

2. The design discharge capacity of 3420 cfs at S-46 was used for the other fixed point on the curve. This 3420 cfs is equivalent to 50% of the Standard Project Flood.

This formula was then used by the Corps of Engineers to calculate design discharges for project culverts along the C-18 canal. Discharge capacity for the C-18 canal east of State Road 710 was based on these culvert discharges. In the permit process,

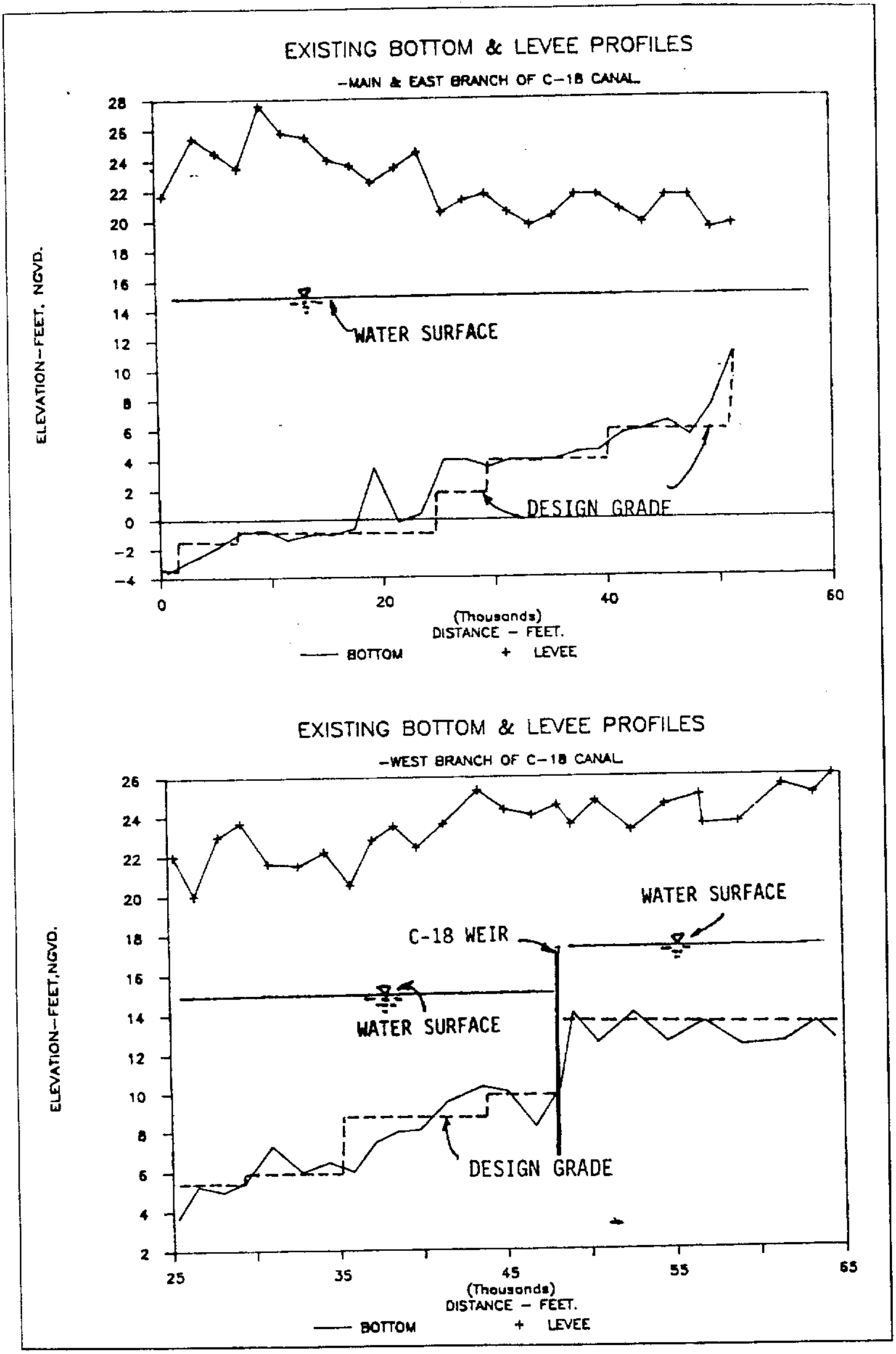

FIGURE 5. Existing Bottom and Levee Profile Along C-18

subbasins in the area have typically been subdivided into parcels (based on the pending application). The formula was applied to each parcel independently to calculate a permissible discharge. The formula, however, tends to assign a higher discharge per unit area when a small area is considered. As a result, the subbasin as a whole is permitted a greater discharge per unit area than that which was used to design the subbasin's outfall culvert.

Table 3 presents permitted and allowable runoff for each subbasin (see Appendix A for a list of permitted areas).

D. Subbasin Descriptions

The entire C-18 basin was divided into 21 subbasins. The drainage from these subbasins to C-18 is generally via culverts with risers. There are a number of local private landowners and subdivisions operating smaller pumps and water control structures as part of the secondary drainage system. Table 4 presents an inventory of the C-18 secondary structures. The topography and land uses are presented in Figure 3 and Table 2.

1. Subbasin 1. This basin includes the eastern portion of J. W. Corbett Wildlife Management Area. This basin is very flat and subject to long duration flooding. Current drainage of the basin is via two 72 inch corrugated metal pipe (CMP) culverts with flashboard risers, with crest elevations at 17.7 ft NGVD. The flashboard risers are in a deteriorated condition. Staff gage readings upstream of the culvert are available since 1976. The highest reading, 23.4 ft NGVD, occurred on June 23-24, 1982. The permitted discharge for this basin is 1/4 inch per 24 hours. (Permit No. 50-00251-S). (See Appendix A)

2. Subbasin 2. This basin has an area of 7,115 acres owned by Pratt & Whitney for their Grovernment Products Division and Engine Development and Testing site. About 90 percent of the basin can be considered wetland. The present drainage system includes a 50 cfs pump facility, three water control structures, and a canal system. Approximately one-third of their runoff is discharged into C-18 via two 72 inch CMP culverts with risers (crest elevation 21.5 ft NGVD). The rest of the runoff discharges to the Corbett Wildlife Area. The total runoff allocation for this basin is 2.35 inches per 24 hours.

3. Subbasin 3. This basin has an area of 2,240 acres of open land within the Indian Trail Water Control District. A conceptual drainage permit application for this area was submitted in August 1979.

TABLE 3. COMPARISON OF CURRENT RUNOFF ALLOCATIONS IN THE C-18 BASIN(3)

GDM Design Discharges-cfs			SFWMD Permit Discharge, cfs	
Subbasin Number	Station-Ft.	Accumulated	Permit(1)	Permit(2)
West Branch				
1	413+00	0	118.0	118.0
2	403+00	0	757.0	319.0
4	367+00	0	80.6	84.0
3	347+00	190	91.0	80.8
5	242+00	190	34.0(2)	34.0
6	241+00	190	45.0	45.0
7+8	231+05	390	1010.0	1035.0
9	181+70	940	142.8(2)	142.8
10(6)	105+10	1440	381.5	431.2
East Branch				
11	303+00	-	22.2(5)	22.2
12(6)	207+60	300	419.5	521.3(4)
13	191+79	650	342.0(4)	342.0(4)
14+15(6)	147+24	990	308.8(4)	362.9(4)
16+17(6)	40+30	1350	177.0(4)	177.0(4)
Junction	252+96	2790	-	-
18+19(6)	193+15	3240	-	384.0
21	132+00	3240	96.3(2)	96.3
20	76+20	3420	111.0	111.0

(1) Based on SFWMD permit file (Appendix A).
(2) SFWMD runoff criteria of 1 inch per 24 hours west of Beeline Highway and $Q = (114/\sqrt{A} + 34)*A$ for the area east of the Beeline Highway.
(3) Not all permitted discharge can be achieved due to higher tailwater conditions in the C-18 canal.
(4) Assume no additional discharge from proposed water management areas.
(5) Existing pump capacity.
(6) A total allowable discharge of 610 cfs from the proposed water management area is not included in the calculation.

TABLE 4. C-18 SECONDARY STRUCTURES

Sub-basin Number	Approx. Sta. Ft.	No. of Pipes	Pipe Diam. Inches	Pipe Length Feet	Top Elev. of Flashboard Ft. NGVD	Length of Riser Feet	Invert of Pipe Ft. NGVD	Remarks
West Branch								
1	413+90	2	72	45.0	17.70	6.0	14.80	
2	403+40	2	72	87.0	21.5	12.0	14.60	
3	395+00	1	60	102.0	19.50	5.0	13.20	
3	375+00	1	60	102.0	-	-	14.00	Plugged
3	347+00	1	60	102.0	19.50	5.0	13.28	
4	367+00	1	54	84.0	-	-	24.70	No board
4	366+60	2	30	79.0	24.40	-	17.25	No board
5	323+90	1	36	43.0	24.40	-	16.13	Plugged
5	312+75	1	36	43.0	26.10	-	16.28	Plugged
5	300+90	1	36	43.0	24.60	-	16.04	Plugged
5	281+65	1	36	33.0	25.10	-	17.13	Plugged
5	274+40	1	36	40.5	-	-	16.00	Plugged
5	261+00	1	36	39.0	-	-	14.92	Plugged
5	249+10	1	24	42.0	-	2.0	15.80	
5	242+10	2	54	46.0	18.10	4.5	14.70	
6	241+00	1	42	42.0	18.30	3.5	15.20	
7	231+05	3	72	64.0	18.00	65.0	-	
8	231+10	1	48	60.0	18.00	7.0	11.50	
9	181+70	4	66	59.0	14.10	22.0	11.10	
10	105+10	4	66	55.0	-	22.0	10.60	
East Branch								
11	303+00	1	48	-	-	-	-	Pump
12	207+60	3	72	55.0	16.90	18.0	8.20	Top board elevation 16.3-17.2
13	252+79	1	12	20.0	-	-	17.35	
13	235+99	1	48	63.0	18.10	78.0	-	
13	209+49	1	72	50.0	18.10	145.0	-	
13	191+79	1	72	50.0	18.20	145.0	-	
14	147+24	1	72	55.0	16.90	6.0	7.80	
15	148+39	1	72	50.0	-	6.0	15.90	No board
15	148+29	2	72	50.0	-	12.0	10.02	No board
16	44+50	1	72	56.0	16.53	6.0	8.20	
17	40+30	2	72	50.0	-	12.0	9.00	No board
C-18 Main								
18	193+15	3	72	53.0	-	18.0	9.20	No board
19	193+15	1	72	59.0	-	6.0	8.40	No board
SIWCD	147+50	1	54	100.0	-	-	-	Screw gate
20	111+35	1	36	60.0	-	-	10.60	Pumped
20	111+35	2	48	70.0	-	-	7.50	Pumped
20	76+20	1	48	58.0	-	-	9.20	Pumped
21	132+00	1	42	48.0	*13.0	-	10.25	Poor
Fl Tpke	43+20	1	36	38.0	*13.0	-	10.80	No board
Fl Tpke	42+50	1	36	52.0	*13.0	-	8.20	No Board

*Based on Table 7, "Draft Feasibility Report and Environmental Assessment C-18 - Loxahatchee Slough Central & Southern Florida" June 1983, Jacksonville District, U. S. Army Corps of Engineers.

Extensive drainage network has been constructed in the basin. The current storm runoff is drained into C-18 via three 60 inch CMP culverts with risers (crest elevation 19.5 ft NGVD) at three locations. The middle culvert has been plugged by an earth dam.

4. Subbasin 4. This basin, known as Ranch Grove, comprises 1,919 acres. An extensive canal network exists which maintains the water level at 18.4 ft NGVD. The runoff is pumped into a 300 acre above-ground retention reservoir. The runoff is released into C-18 via a 54 inch CMP culvert with 84 inch riser (crest elevation at 24.7 ft NGVD and bleed down to 24 ft NGVD) . The permitted discharge for this basin is 1 inch per 24 hours.

5. Subbasin 5. This basin comprises 820 acres. There is no drainage permit on file. There are five 36 inch CMP culverts, one 24 inch CMP culvert with riser, and two 54 inch CMP culverts with risers. However, the five 36 inch culverts have been plugged.

The two 54 inch CMP culverts with risers were provided for the drainage of the Seaboard Coastline Railroad (SCL RR) and SR710. Overall, the drainage in this basin is rather poor.

6. Subbasin 6. This basin is an 1,137 acre portion of North Palm Beach County Water Control District Unit 8. The existing agricultural land adjacent to the west leg of the C-18 canal has drained into this reach via a 42 inch CMP culvert with riser (crest elevation at

18.3 ft NGVD). The rest of Unit 8 is Subbasin 12B. Approximately two-thirds of subbasin 6 can be considered wetland. A discharge of 45 cfs, which is 1 inch per 24 hours over the basin, has been permitted by the District.

7. Subbasin 7. This basin comprises an area of 14.5 square miles (9,291 acres). This includes 1,600 acres of residential area in Caloosa, Palm Beach Park of Commerce (PBPC) (1,880 acres), and additional parcels to the north and west. The PBPC is to be developed into a residential and industrial complex. The drainage of the PBPC is via a 40 ft weir with a crest elevation of 18.2 ft NGVD and bleed down to 18.0 ft NGVD, to the main canal system of the Caloosa development. At present there is minimum drainage for the acreage north and west of PBPC during flood conditions until the gravity head becomes available in the PBPC and Caloosa system. The drainage system for Caloosa drains into the C-18 canal at a point downstream of the C-18 weir. The total permitted discharge is 927 cfs for the entire subbasin which is about 2.4 inches per 24 hours. The outfall of the basin includes three 72 inch culverts through the north right of way of the C-18 canal, and a 65 ft wide sheet pile weir immediately upstream of the culverts with a crest elevation at 18.0 ft NGVD and bleed down to 16 ft NGVD by a 2 ft x 2 ft opening.

8. Subbasin 8. This basin, owned by Fox Trail Inc., comprises 243 acres of low density residential area. The drainage system includes a canal/lake network and a 48 inch CMP culvert with 84 inch riser. Permitted discharge is 83 cfs (8.2 inches per 24 hours). The crest elevation of the riser board is 18 ft NGVD and bleed down to 16 ft NGVD by a 1 inch slot. The water level in the internal canal system is maintained at 16 ft NGVD.

9. Subbasin 9. This subbasin comprises 604 acres of low density residential area. The drainage system which was constructed prior to 1970 is not under surface water management permit. The outfall structure of the basin is by gravity with a four 66 inch CMP culverts with risers (top board elevation at 13.7 to 15 ft NGVD). During severe storm conditions, these culverts allow free exchange of runoff between C-18 and the subbasin due to high tailwater conditions.

10. Subbasins 10, 14, and 16. These basins comprise approximately 5,718 acres of agricultural land and wetland, and are a portion of the 16,575 acres of land owned by the McArthur Foundation within the C-18 basin. There is no physical divide existing between subbasins 10, 14, and 16. The subbasin alignment proposed by the MacArthur Foundation (Permit No. 50-01626-S) was used in this study to separate subbasins 10, 14, and 16. This alignment closely follows an existing drainage ditch on the east side of subbasin 10. The natural slope of the land declines from an elevation of 20 ft NGVD in the west to an elevation of 16 ft NGVD in the east. Accordingly, overland flow from subbasin 10 can contribute to the east leg of C-18 canal when the water levels in subbasins 14 and 16 are low. Drainage ditches exist within subbasin 10 and drain to the west leg of the C-18 canal via four 66-inch CMP culverts with risers (top board elevation at 15.18 to 16 ft NGVD). Under MacArthur Foundation's conceptual permit, the runoff from subbasin 10 is allowed to be discharged at 396 cfs into a proposed water management area (subbasins 14 and 16).

11. Subbasin 11. This basin comprises an area of 350 acres of agricultural land. The area is drained by a 10,000 GPM pump discharging into the southern end of the east leg of the C-18 canal.

12. Subbasin 12. As mentioned previously, this basin was divided into two subbasins (i.e., 12A and 12B). Subbasin 12A is bounded by PGA Boulevard on the north, SR 710 on the south and west, and C-18 on the east. Subbasin 12A consists of about 2,196 acres of wetland (also part of the original Loxahatchee Slough). The drainage of this area is by three 72-inch culverts with risers and discharges into the east leg of C-18 canal. The risers were constructed as part of the project design to prevent over drainage. The risers were not maintained and had deteriorated. The risers were repaired and the water level in subbasin 12A was raised to 16.90 ft NGVD in the late 1970s or early 1980s by the Corps. A feasibility and environmental assessment study in the C-18 basin and the Loxahatchee Slough was completed by the Corps (1981). Results show that increased water levels in the Loxahatchee Slough have been beneficial to the wetlands and provided other water resources benefits. Failure to maintain high water levels in subbasins 12A, 14, and 16 has, in the past, overdrained these marsh areas. In 1982 the District decided to install boards in the culverts located in subbasins 14 and 16 to an elevation of 16.90 ft, and 16.53 ft NGVD, respectively. Since then, the water level in these marsh areas has been maintained above the normal water level of 14.80 ft NGVD in C-18.

Subbasin 12B does not have a positive outfall to the east leg of the C-18 canal. The area, however, has been allocated 1 inch per 24 hours of runoff rate (Permit No. 50-00037-S). The future drainage of this basin requires the runoff being discharged into the proposed water management area in subbasin 12A prior to discharging into the C-18 canal.

13. Subbasin 13. This basin comprises a 2,427 acre portion of the PGA Community including about 2,100 acres of residential area, and the rest wetland. The water management system includes pumps and a canal/lake network. The pumps are used to maintain optimum water levels of the canal/lake system to an elevation of 15.0 ft NGVD. A total pump capacity of 342 cfs (about 3.9 inches per 24 hours) is allowed to pump runoff from the community into the wetland outside the PGA Resort Community and drain into the C-18 canal via one 48 inch CMP culvert with a 52 ft long sheet pile weir, and two 72 inch CMP culverts with a 119 ft long sheet pile weir. The crest elevation of these weirs is 18.2 ft NGVD with a bleed down at 18 ft NGVD. The community is protected by a berm to prevent backwater from the wetland area.

14. Subbasins 15 and 17. Subbasin 15 comprises an area of about 3,146 acres of wetland, agricultural, and urban land uses (Table 2). The MacArthur Foundation has received a surface water management conceptual approval from the District for 2,549 acres and the remainder to be included in the proposed water sanctuary (Permit No. 50-01626-S). There is an extensive canal/lake system within the basin. The runoff is drained into the east leg of C-18 canal via three 72 inch CMP culverts with risers (currently no boards in place). As a result, these culverts allow free exchange of runoff between C-18 and the subbasin during severe storms. The canal system is part of the Northern Palm Beach County Water Control District (NPBCWCD) system. This area has been overdrained. Any future water management plan to alleviate this overdrainage must consider the possible impact to the drainage system serving developed areas such as Old Marsh and East Pointe (subbasin 17).

Subbasin 17 is a developed residential area including Eastpointe and Old Marsh. East Pointe subdivision is a development of 640.73 acres bounded on the north by Donald Ross Road, on the south by Hood Road, on the east by the Florida Turnpike, and extends west from the Florida Turnpike approximately 6,000 feet. The general ground elevation is about 18.5 ft NGVD and is very flat. There is an extensive surface water management system made up of pumps, canals, and lakes. The allowable discharge rate during a 25 year design storm is 148 cfs (Permit No. 50-00532-S). An existing pump station with two 30,000 GPM pumps and one 2,000 GPM pump, discharges excess stormwater into a tributary outfall canal into the NPBCWCD canal, as mentioned previously. The desired normal water level in the canal/lake system is 15.0 ft NGVD, but this water level has not been maintained in the last few years. The lower water levels may have been caused by the Seacoast Utilities wellfield located east of the area. There is a need to recharge the groundwater table and maintain adequate water for the area as well as restoration of the original Loxahatchee Slough west of this subdivision. Old Marsh is comprised of an area of about 446.4 acres of residential development located to the west of the Eastpointe subdivision. The surface water management system includes one 7,500 GPM pump, and one 5,500 GPM pump discharging to the east leg of C-18 canal via an existing NPBCWCD drainage ditch (Permit No. 50-01411-S). As discussed previously, the marsh area in this subbasin has been overdrained. The District is investigating impacts of the installation of flashboards on the existing project culverts to raise the water level.

15. Subbasin 18. This basin comprises 2,062 acres of wetland. An area of about 1,065 acres was proposed to receive runoff from a 300 cfs pump from the South Indian River Water Control District and 40 cfs from the area to the west. (Permit No. 50-01626-S). The existing outfall system for this subbasin drains into the C-18 canal via three 72 inch CMP culverts with risers (currently no boards in place). Ground elevation ranges between 14 ft to 20 ft NGVD and the C-18 canal stage is normally 14.8 ft NGVD. As a result of the low land elevations and high tailwaters downstream, this subbasin receives offsite runoff during severe storm conditions.

16. Subbasin 19. This subbasin comprises 2,140 acres of wetland (part of the original Loxahatchee Slough). Under the approved conceptual plan proposed by the MacArthur Foundation, an area of about 1,817 acres will be managed by a surface water management system, and the rest of the basin will be preserved as a water management area (Permit No. 50-01626-S). The runoff rate allowed under a 25 year design storm is 220.2 cfs into the proposed water management area. The existing outfall system for the basin consists of one 72 inch CMP culvert with riser (currently no boards in place). Because there are no boards in the riser, this basin receives inflow from the C-18 canal during high tailwater conditions.

17. Subbasin 20. This basin comprises 507 acres of agricultural land. An extensive drainage network consisting of pumps and retention areas drains into the C-18 canal via a 48 inch CMP culvert with riser (currently no boards in place). The combined allowable discharge for the basin is 111 cfs (6.36 inches per 24 hours).

18. Subbasin 21. This basin comprises 313 acres. Approximately two-thirds of the area can be considered wetland, and one-third as urban land use. The eastern portion of the basin is part of the South Indian River Water Control Distruct. Due to the

nature of the ground elevation, this portion of land is currently drained into a wetland prior to discharging into the C-18 canal via a 42 inch CMP culvert with riser. The drainage for this area has been poor, and the existing culvert requires some maintenance. The wetland portion of this basin was part of the original Loxahatchee Slough.

III. METHODOLOGY

A standard hydrologic approach was applied according to the following procedure: a) design rainfall events of selected frequencies were determined for the drainage basin; b) rainfall excess was computed by using the SCS curve number method; c) unit hydrographs were calculated for each subbasin; d) design runoff was determined for each subbasin by the application of rainfall excess to the unit hydrograph of each subbasin, and the outflow hydrograph for each subbasin was routed to the main channel according to the limitations of the existing and/or permitted outlet structures; e) design hydrographs of the subbasins were combined and routed downstream to the outlet point of C-18 by the dynamic wave routing procedure.

A. Design Rainfall and Time Distribution

The selection of maximum one-day rainfalls for the 1-in-10, 1-in-25, and 1-in-100 year storm events was based on the SFWMD Technical Publication 81-3 (MacVicar, 1981). This one-day maximum rainfall was further distributed into a 3-day rainfall event according to the stormwater standards of SFWMD (Volume IV, Permit Information Manual). The remaining daily rainfall distribution was based on the rainfall measured during September 21-October 19, 1960, except the fourth and fifth day rainfall were replaced by the first and second day of the observed event (Table 5) which is consistent with the Corps studies.

Historical storm events presented in the C-51 basin study (Water Management Planning For The Western C-51 Basin, March 1984) indicated that the SCS Type II distribution provided an adequate representation for design rainfall distribution in this general area. Therefore, the Type II distribution, which is typically used to represent regions of high rate runoff resulting from summer thunderstorms, was used in this study.

B. Rainfall Excess of the SCS CN Method

1. Subbasin Delineation. The subbasins of C-18 were defined based on a survey of all existing inflow points along the C-18 canal and the SFWMD permits. Figure 2 presents the 21 subbasins which have been delineated for this study.

TABLE 5. THIRTY DAY RAINFALL DISTRIBUTIONS - 10, 25, AND 100 YEAR STORMS

Day	Sept. 1960 Storm	10-Year	25-Year	100-Year
1	0.43	1.10	1.31	1.68
2	0.50	1.60	1.92	2.45
3	3.96	7.50	9.00	11.50
4	0.71	0.43	0.43	0.43
5	2.07	0.50	0.50	0.50
6	0.18	0.18	0.21	0.27
7	0.25	0.25	0.30	0.38
8	0.30	0.30	0.36	0.46
9	0.03	0.03	0.04	0.05
10	0.11	0.11	0.13	0.17
11	0.00	0.00	0.00	0.00
12	0.00	0.00	0.00	0.00
13	0.00	0.00	0.00	0.00
14	0.00	0.00	0.00	0.00
15	0.13	0.13	0.15	0.20
16	0.31	0.31	0.37	0.47
17	0.39	0.39	0.46	0.60
18	0.20	0.20	0.24	0.30
19	0.72	0.72	0.86	1.10
20	2.01	2.01	2.39	3.08
21	0.00	0.00	0.00	0.00
22	1.00	1.00	1.19	1.53
23	0.46	0.46	0.55	0.70
24	0.59	0.59	0.70	0.90
25	0.00	0.00	0.00	0.00
26	0.00	0.00	0.00	0.00
27	0.00	0.00	0.00	0.00
28	0.23	0.23	0.27	0.35
29	0.28	0.28	0.33	0.43
30	0.00	0.00	0.00	0.00
	14.96	18.32	21.71	27.55

2. Hydrologic Soil Group. Major soil groups within each subbasin are the B/D hydrologic soil group. In general, the hydrologic soil group of A represents high infiltration rates even when thoroughly wet, and consists chiefly of deep, well to excessively-drained sand or gravel. Group D has a very slow infiltration rate when thoroughly wet and, therefore, has high runoff potential. Table 6 indicates that most of the soil in the subbasin has a dual group of B/D. Soil group B has a moderate infiltration rate when thoroughly wet. The dual symbol of B/D indicates the top soil has a moderate rate of transmission, and the subsoil has an extremely poor drainage capability. In the selection of a SCS curve number (CN) for each land use type, the CN value under the C group was used in the computation of a composite CN value for most subbasins, except subbasin 20.

C. Development of Unit Hydrographs

Two basic approaches were used to develop the unit hydrograph and the composite (runoff) hydrographs for each subbasin. They are, (1) the Tracor procedure combined with the Cypress Creek formula (see Appendix E) and (2) the HEC-1 stream network model. The modified Tradcor procedure (1968) was applied to develop the 30-minute unit hydrograph.

In the application of the HEC-1 program, the subbasin was further divided into two or more according to their land uses. The estimated SCS curve number, and time of rise for the unit hydrograph as mentioned previously, were input into the stream network model of the HEC-1 program, including routing through existing flashboard risers or spillway to obtain the basin outflow hydrograph (see Appendix E for detail).

D. Composite Hydrograph.

The composite hydrograph for each subbasin was computed by multiplying the ordinate of the unit hydrograph by successive runoff increments, and summing up the partial hydrographs. This composite hydrograph was then routed through existing pumps or storage areas into the C-18 canal via the existing culverts at each subbasin.

E. Hydraulic Routing - EPA EXTRAN Model

During major rainfall events, the flat lowland basins and their associated outfall structures of the secondary drainage systems are frequently submerged due to high tailwater conditions in C-18. Such basins are subject to inflows from the canal before offsite discharge begins. In order to model such a situation, a hydrodynamic wave routing model is required. EXTRAN is a hydraulic flow routing model for open channel and/or closed conduit systems. The EXTRAN model receives composite subbasin runoff hydrographs at their outlet locations, and performs dynamic routing of stormwater flows through the major storm drainage system to the points of outfall to the receiving water system. The model can simulate branched or looped networks, backwater due to tidal or nontidal conditions, free-surface flow, pressure flow,

TABLE 6. PHYSICAL CHARACTERISTICS OF THE C-18 BASIN

Subbasin No.	D.A. (acres)	L Miles	Slope Ft/Ft	Hydrologic Soil Group	S Inches	Remarks
1	13378	12.50	0.000023	B/D	2.99	
2	7115			B/D	1.23	4 subbasins
3	1920	3.22	0.000088	B/D	2.66	
4	1901			B/D	2.50	2 subbasins
5	820	1.70	0.00017	B/D	2.99	
6	1137	2.84	0.000067	B/D	2.90	
7	9291			B/D		2 subbasins
8	273	1.33	0.00014	B/D	2.50	
9	604	1.90	0.00002	B/D	2.66	
10	3288	3.60	0.000131	B/D	3.61	
11	350	1.61	0.00012	B/D	3.51	
12A	1207	2.27	0.000083	B/D	1.36	12B not contributed
13	2426	0.76	0.00025	B/D	2.50	2 subbasins
14	960	2.46	0.000154	B/D	1.42	
15	2746	4.73	0.00008	C	3.61	
15A	399			B/D	2.82	
16	1478	1.89	0.0001	B/D	1.36	
17	1140			B/D,C	2.90	3 subbasins
18	2062	3.03	0.00019	B/D	2.99	
19	2139	2.27	0.000125	B/D	4.10	
20	506	1.70	0.000222	A/D	3.89	use soil group B
21	312	1.42	0.000133	B/D	1.49	

flow reversals, flow transfer by weir, orifice and pumping facilities, and storage at on-line or off-line facilities. Types of channels that can be simulated include circular, rectangular, horseshoe, eggs, basket handle pipes, and trapezoidal channels. For a detailed description, model performance, assumptions, and routing theory, refer to the Stormwater Management Model User's Manual, Version III, Addendum I EXTRAN by Larry A. Roesner, Robert P. Shubinski, and John A. Aldrich, of Camp Dresser & McKee, Inc., January 1983. A description of details on the model's application in the study is presented in Appendix F.

Figure 6 presents the system schematic which was applied to the modified EXTRAN model for this study. The 10, 25, and 100 year design rainfalls were input to the basin model. Composite subbasin hydrographs, based on either the C-51 basin procedure or the stream network model of the HEC-1 program, were input into the channel network system of the modified EXTRAN model to route the flood water downstream to S-46, the outlet structure of the C-18 basin.

F. Peak Flood Stage Estimation for Subbasin

The following steps were taken to develop the flood stage in each subbasin:

1. Establish a stage-storage relationship based on the latest available topographic data and permit information (Appendix D).

2. Compute total volume of water that enters the basin as rainfall and subtract the maximum soil moisture storage available for the basin.

3. Compute volume of runoff remaining in each subbasin at the end of each day by subtracting basin outflow from the net rainfall input to the basin.

4. The net remaining volume of runoff in the basin is then converted to elevation in feet NGVD, based on the stage-storage relationship developed in Step 1.

IV. DISCUSSION OF RESULTS

The peak hours for the flood profiles to reach their maximum height is different for the channel reaches downstream and upstream of the C-18 weir. The time to reach the peak stage at the C-18 weir is also different from the west end of C-18 near the Corbett Wildlife Management Area. This is due to the occurrence of submerged conditions at the C-18 weir during the peak flood period. In general, the backwater profile upstream of the the C-18 weir reached its peak at a much later time than the downstream reach. Figures 7, 8, and 9 present the backwater profiles along the main and east branch of C-18 for 10-, 25-, and 100-year design storm events. Figures 10, 11, and 12 present the backwater profiles along the main and west branch of C-18 for 10-, 25-, and 100-year design storm events. Two backwater profiles are presented for the reach upstream of C-18 weir. The second profile which is slightly higher than the first one reached its peak at a much later time after the peak had receded in the reach downstream of the C-18 weir.

A. The 10-,25-, and 100-Year Design Floods

Table 7 compares the maximum discharge computed in this study with the original design discharge (1956 GDM), and with the 1981 Corps study, at each subbasin outfall along C-18 under 10-, 25-, and 100-year design storms. The maximum discharge shown for the reach upstream of the C-18 weir was based on the peak profiles when the peak hours of the downstream reach had already passed, as discussed previously. The discharge values resulting from this study are comparable with the 1981 Corps study except for the area at both ends of C-18. These differences are within 200 cfs under most conditions. The major difference is in the estimation of inflow from the Corbett Wildlife Management Area, and subbasin 11, located at the south end of the East Branch. No contribution of runoff from the West Palm Beach water catchment area to C-18 was assumed in this study, and the outflow from subbasin 11 was limited by the 10,000 GPM pumping station (22.2 cfs) as compared to 240 cfs or more, used in the 1981 Corps' study. Since that study, most of the discharge from subbasin 21 has been diverted downstream of S-46. This modification was taken into account in the present study. These differences in input, along with different methods used in the development of basin hydrographs and hydraulic routing, contributed to the differences in the results.

The peak discharges at the C-18 weir, shown in Table 7, were based on the maximum discharge which occurred at a much later time when the downstream peak was over. The discharge at the C-18 weir would be much less during downstream peak hours. The corresponding discharges would be 204 cfs, 289 cfs, and 413 cfs for 10, 25, and 100 year design storms respectively. The 204 cfs slightly exceeded the design capacity of 190 cfs for this reach of C-18 upstream of the C-18 weir.

B. Water Level Elevations

Table 8 compares the computed flood stages along C-18 from the Corps of Engineers General Design Memorandum (GDM), the 1981 Corps of Engineers

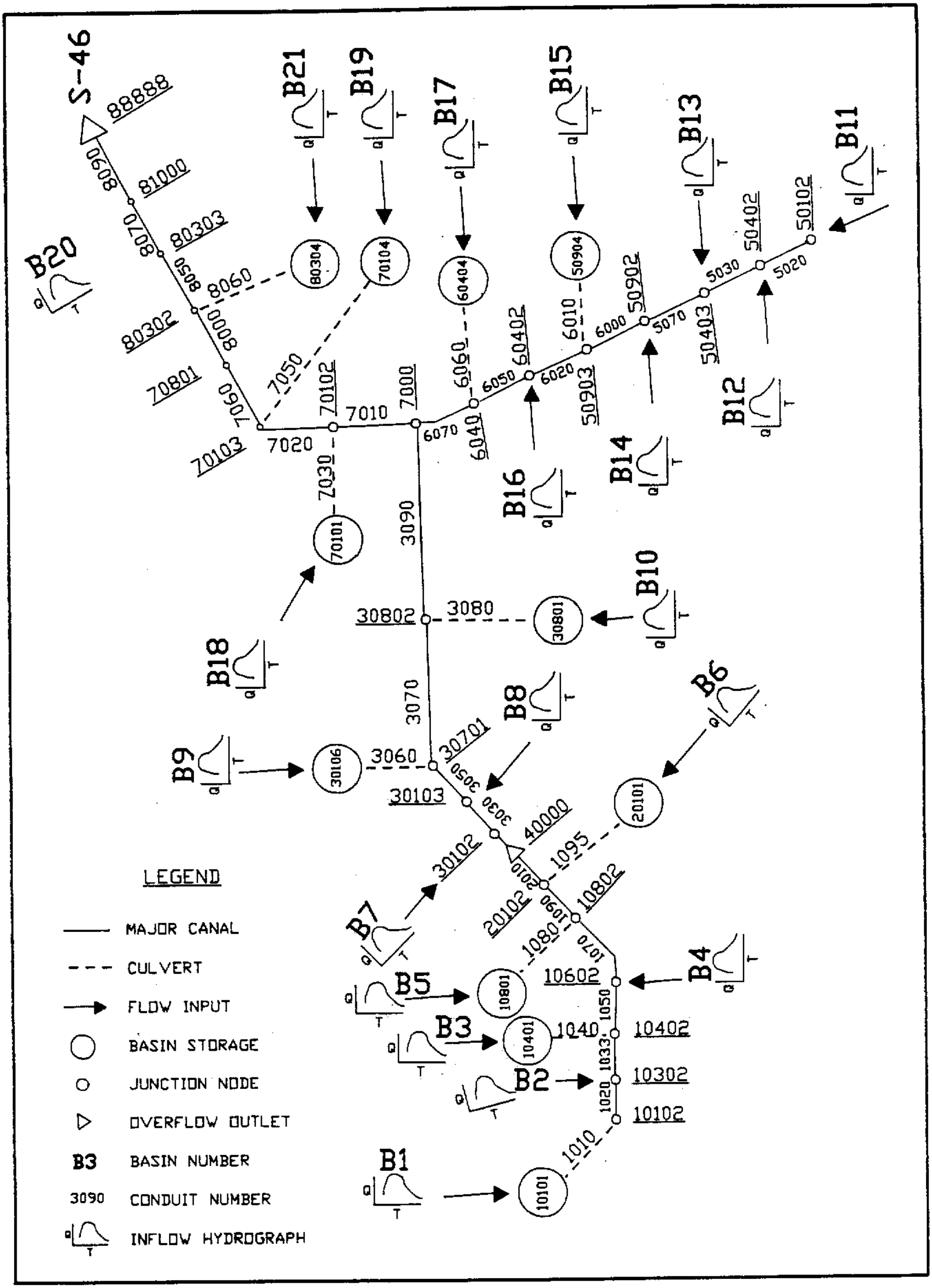

FIGURE 6. Schematic Representation of C-18 and its Secondary Drainage System for usein the EXTRAN Model

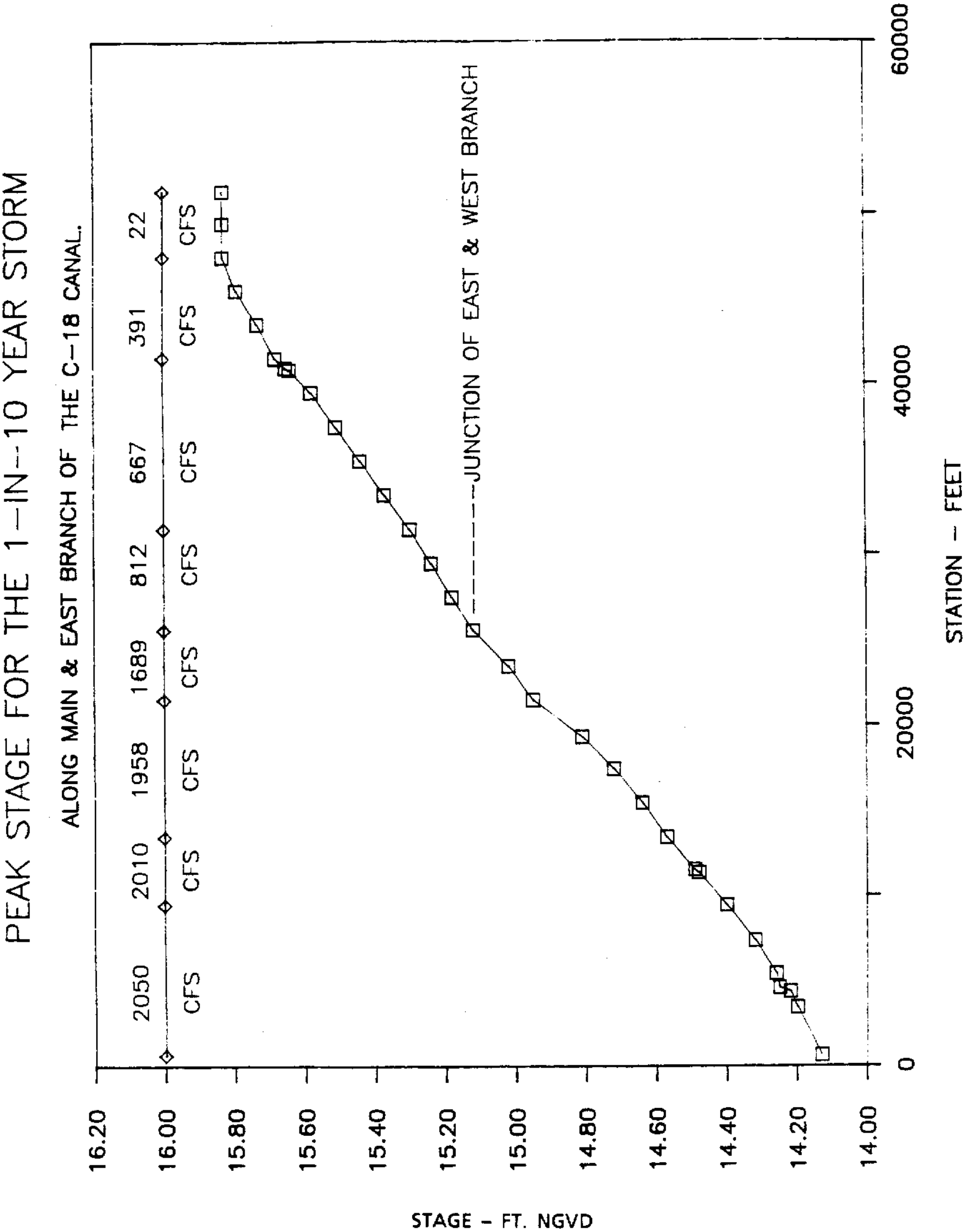

FIGURE 7. Peak Stage for the 1-in-10 Year Storm Along Main and East Branch of C-18Figure 8

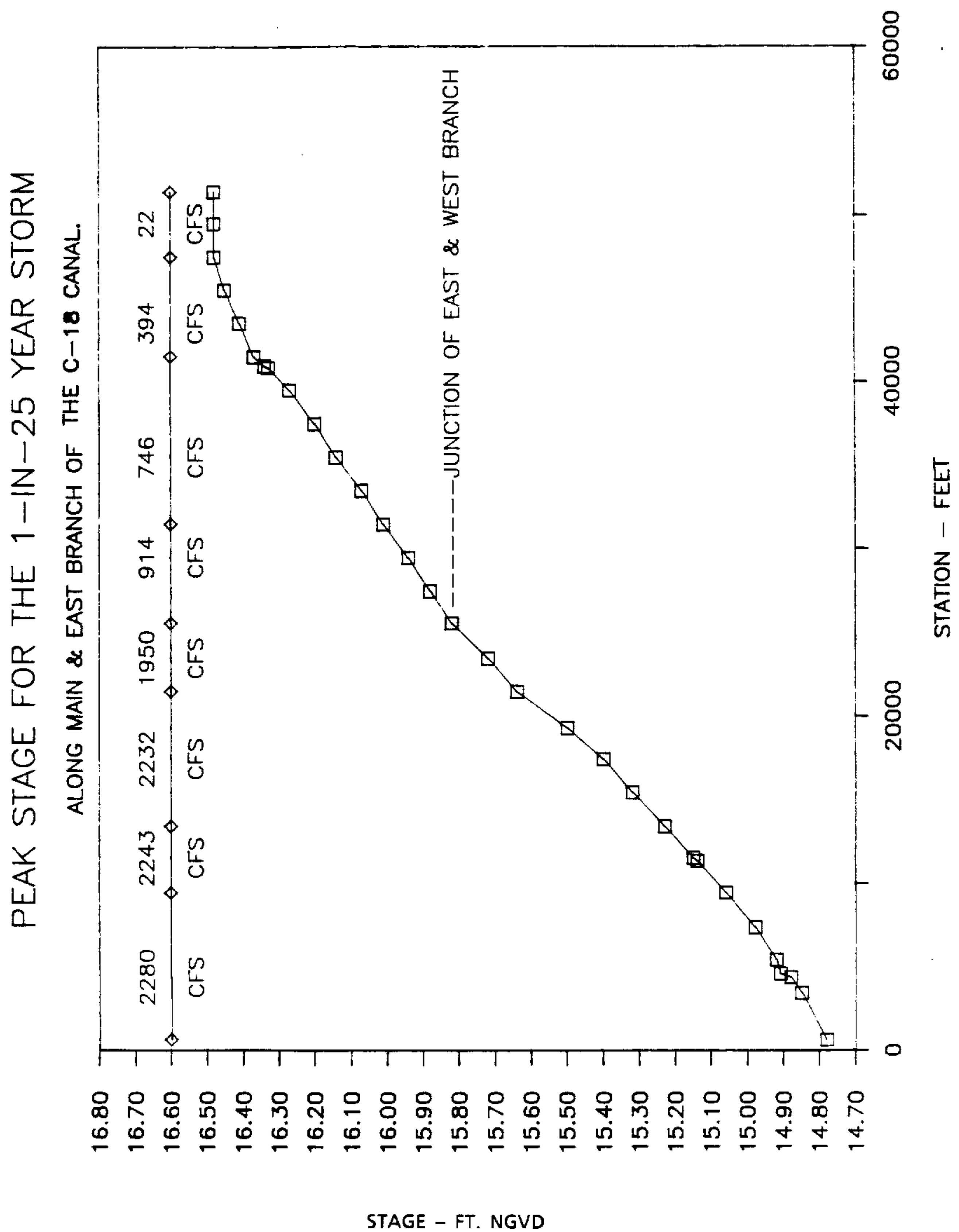

FIGURE 8. Peak Stage for the 1-IN-25 Year Storm Along Main and East Branch of C-18

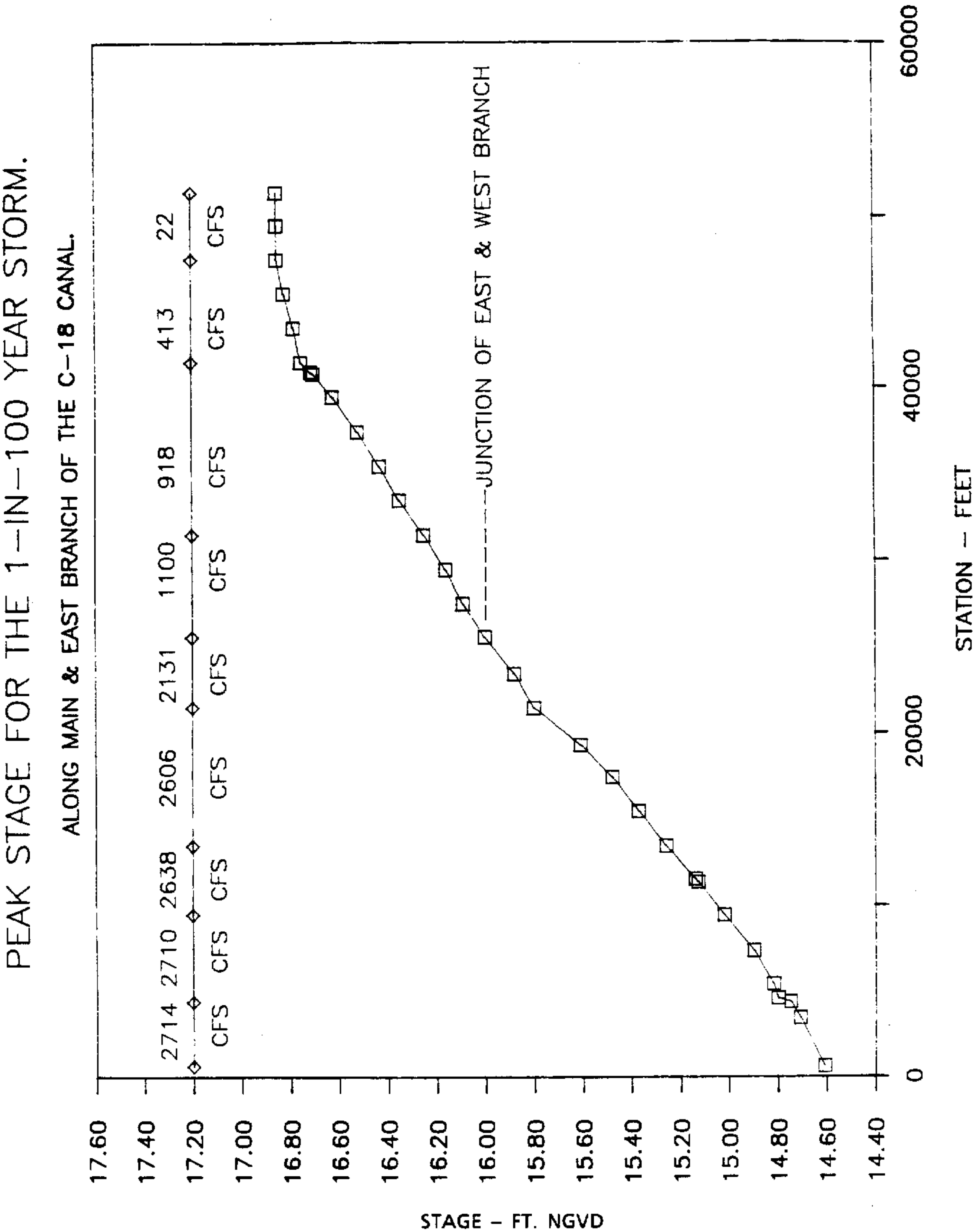

FIGURE 9. Peak Stage for the 1-in-100 Year Storm Along Main and East Branch of C-18

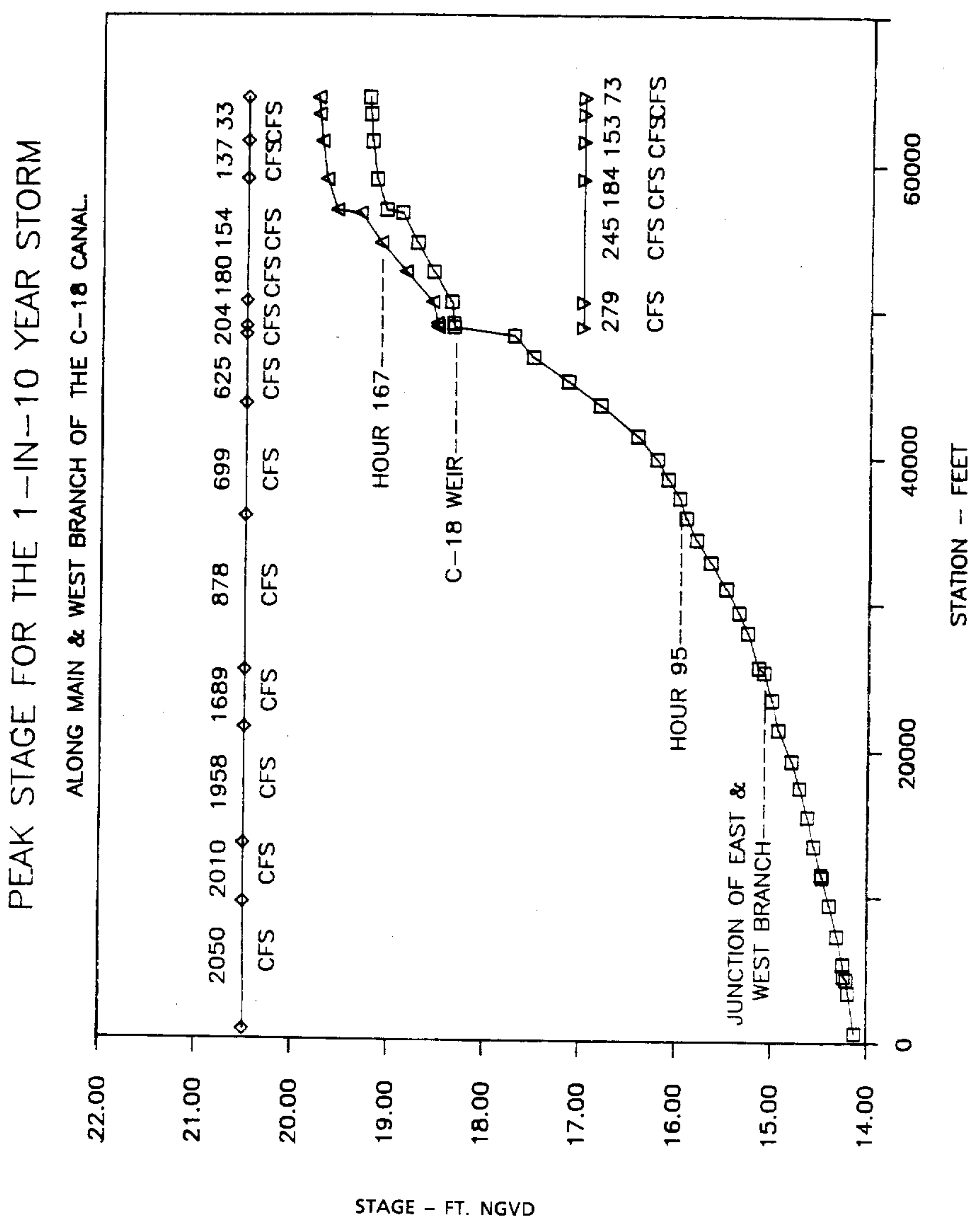

FIGURE 10. Peak Stage for the 1-in-10 Year Storm Along Main and West Branch of C-18

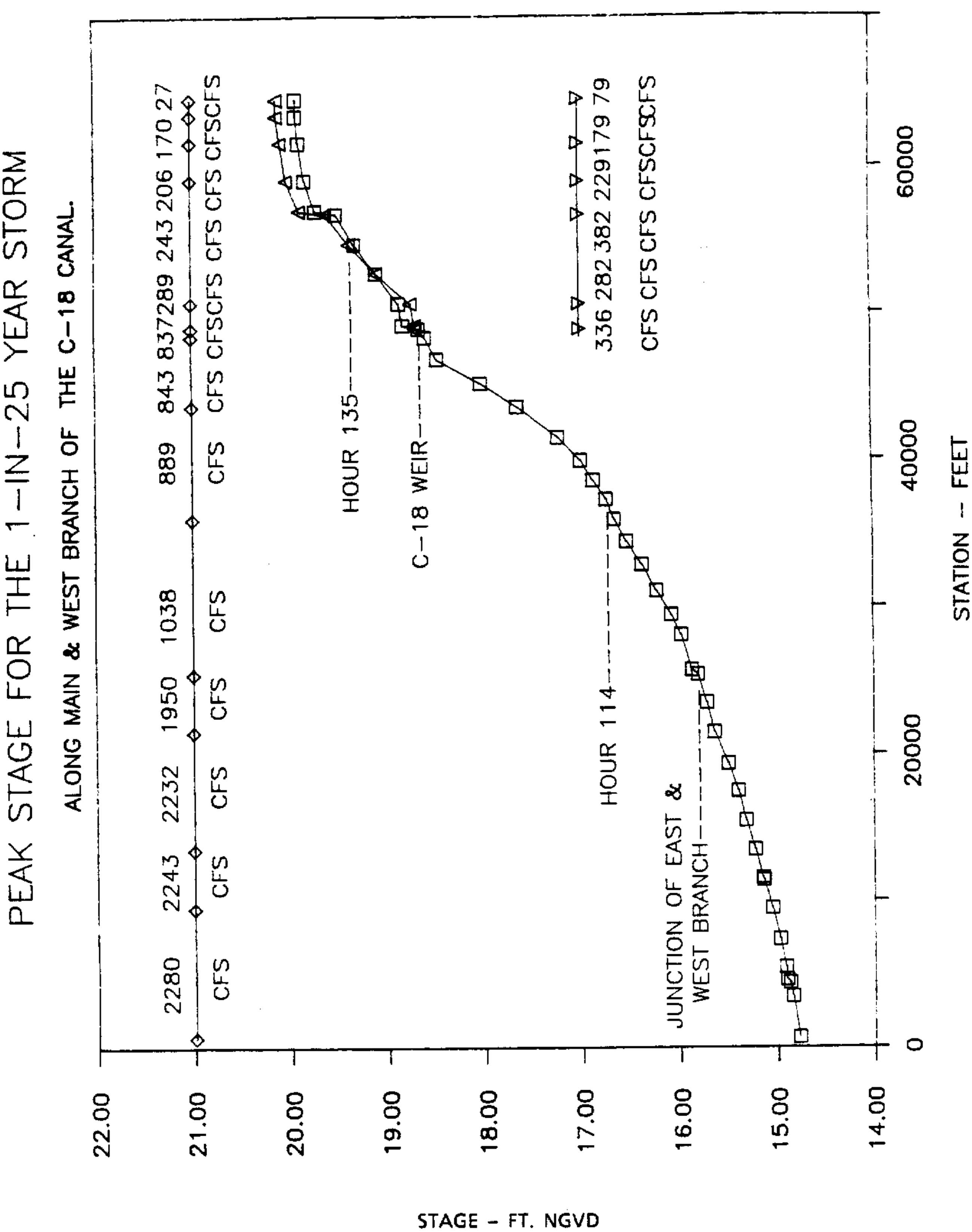

FIGURE 11. Peak Stage for the 1-in-25 Year Storm Along Main and West Branch of C-18

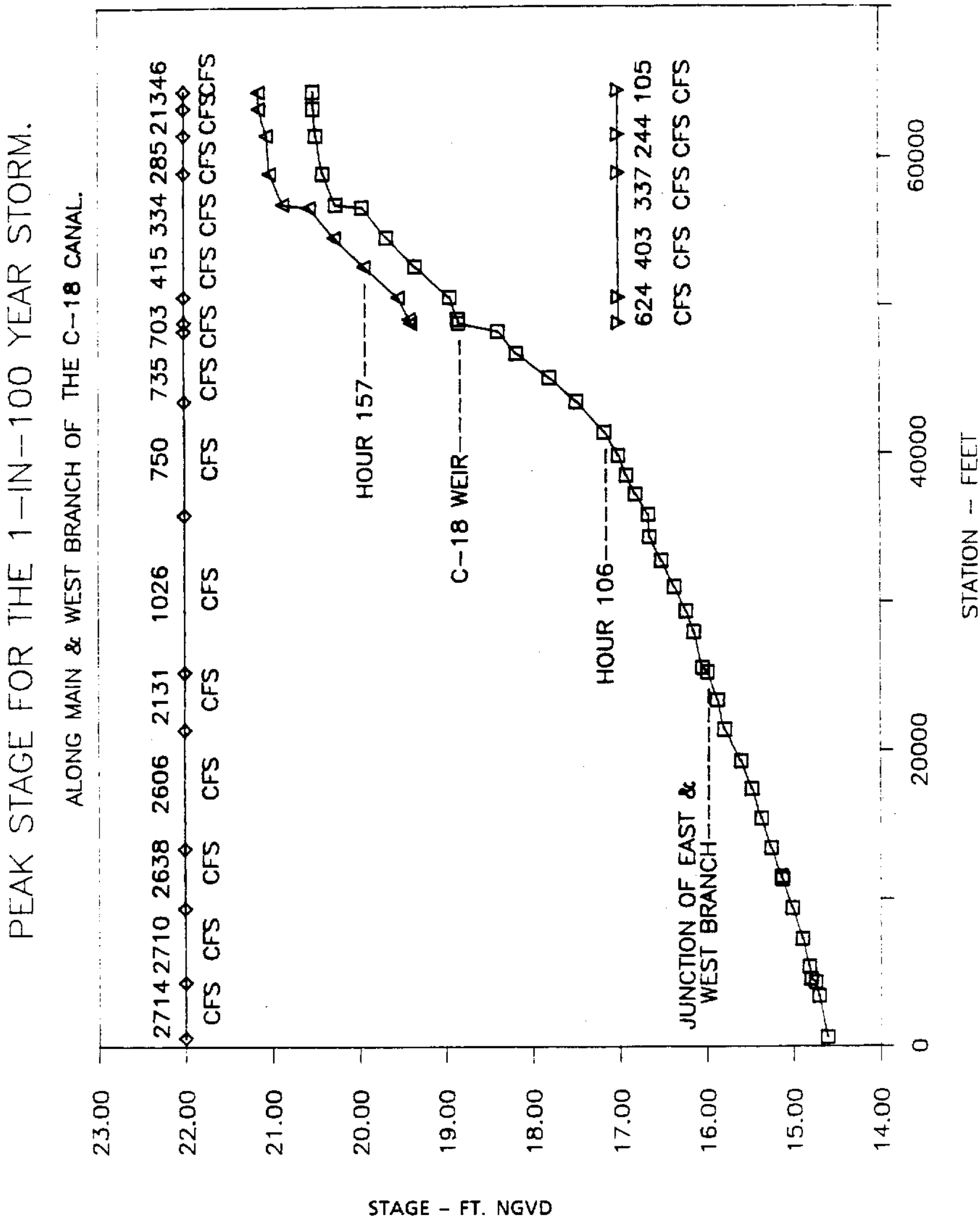

FIGURE 12. Peak Stage for the 1-in-100 Year Storm Along Main and West Branch of C-18

TABLE 7. COMPUTED MAXIMUM DISCHARGE (cfs) ALONG C-18 CANAL

Subbasin Outfall	Station	10-Year WMD 1987	10-Year COE 1981	GDM	25-Year WMD 1987	25-Year COE 1981	100-Year WMD 1987	100-Year COE 1981
West Branch								
1	64553	73	180	0	79	176	105	227
2	63353	73	262	0	79	291	105	331
3	61503	153	322	0	179	351	244	390
4	58953	184	322	0	229	351	337	390
5	50503	245	344	190	282	373	403	412
6	49026	279	367	190	336	397	624	437
weir*	48409	279	367	190	336	397	624	497
7&8	48209	613	747	390	837	796	703	865
9	43496	625	751	940	843	797	750	880
10	35826	699	996	1440	889	1061	1026	1162
Junction	25296	878	996	1440	1038	1064	1026	1163
East Branch								
11	51296	22	240	300	22	297	22	384
12A&13	45396	391	511	650	394	650	413	801
14&15	39396	667	674	990	746	812	918	971
16&17	31396	812	968	1350	914	1113	1100	1288
Junction	25496	1690	1957	2790	1950	2172	2131	2418
18&19	19258	1958	2295	3240	2232	2511	2606	2791
21&22	9396	2010	2332	3420	2243	2550	2638	2826
S-46	0	2050	2332	3420	2280	2550	2714	2826

*Time to peak discharge varied for upstream and downstream of C-18 weir. The corresponding peak discharge for 10, 25, and 100 year design storms at the C-18 weir are 204, 289, and 413 cfs respectively. This occurred when the stage downstream of the C-18 weir peaked.

TABLE 8. STAGE-FLOOD FREQUENCIES - Computed Peak Flood Stage (Ft-NGVD)

Subbasin Outfall	Station	10-Year WMD 1987	10-Year COE 1981	GDM	25-Year WMD 1987	25-Year COE 1981	100-Year WMD 1987	100-Year COE 1981
West Branch								
1	64553	19.77	22.47	21.10	20.11	22.68	21.14	22.95
2	63353	19.76	22.46	21.10	20.11	22.67	21.14	22.93
3	61503	19.73	22.30	21.10	20.07	22.50	21.05	22.75
4	58953	19.68	22.12	21.10	20.01	22.33	21.02	22.58
5	50503	18.57	20.97	20.06	18.76	21.17	19.54	21.44
6	49026	18.52	20.02	19.51	18.69	20.24	19.41	20.56
C-18 Weir	48776	18.51	19.38	18.54	18.66	19.65	19.39	20.03
7&8	48209	17.72	19.35	18.44	18.15	19.61	18.39	19.99
9	43496	16.82	16.39	17.83	17.65	16.73	17.49	17.24
10	35826	15.92	14.76	16.55	16.66	15.17	16.77	15.73
Junction	25296	15.10	13.61	15.20	15.81	14.04	15.99	14.60
East Branch								
11	51296	15.83	15.25	16.28	16.48	15.93	16.85	16.74
12A&13	45396	15.79	14.59	16.28	16.45	15.24	16.82	16.00
14&15	39396	15.73	14.30	16.03	16.27	14.86	16.62	15.55
15&17	31396	15.30	13.81	15.43	16.01	14.27	16.25	14.86
Junction	25496	15.10	13.62	15.20	15.81	14.05	16.00	14.61
18&19	19258	14.81	13.37	14.85	15.50	13.78	15.61	14.32
21&22	9396	14.40	12.26	13.61	15.06	12.58	15.02	13.02
S-46	0	14.13	11.55	12.81	14.78	11.80	14.61	12.15

Note: The stage upstream of C-18 weir reached its peak at a muchlater time than stage at weir and downstream of weir.

study, and the present study. The backwater stages computed by the present study were higher except in the reach upstream of the C-18 weir. The Corps assumed the gates at S-46 would be on manual control and the headwater stage would be held at 12.80 ft NGVD under major storm events.

The major storm condition, as defined in the structure operational manual for S-46, is the condition where the tailwater at the C-18 weir exceeds 17.60 ft NGVD. In this study, the gates at S-46 operate at a rate of 0.4 inch per minute when the tailwater at the C-18 weir is above 17.60 ft NGVD, and the headwater stage at S-46 is above 12.90 ft NGVD. If the tailwater at the C-18 weir falls below 17.6 ft NGVD, or the headwater stage at S-46 falls below 12.90 ft NGVD, then the gates at S-46 close down at the rate of 0.4 inch per minute. As long as the tailwater at the C-18 weir stays below 17.60 ft NGVD, then the gates at S-46 follow the normal operational rule as presented in the previous chapter. Figure 13 presents the stage hydrograph at S-46 under the 10 year design storm event. The peak discharge occurred when the headwater stage at S-46 was 14.78 ft NGVD, not at 12.81 ft NGVD or 11.80 ft NGVD, as assumed by the 1981 Corps study.

The results of this study indicate that S-46 is capable of handling the 100 year design discharge; however, the backwater profile is higher than the original design profile. The western half of the west branch and the reach upstream of the C-18 weir lack the capacity to pass the 10-year discharge according to this study.

C. Peak Flood Stage and Discharge for Each Subbasin

Table 9 presents the maximum flood stage and maximum outflow rate for each subbasin. The estimation of peak flood stage for subbasin 1 (Corbett Wildlife Management Area) was not possible due to lack of survey information. The estimation of peak flood stage for subbasins 2 and 7 was based on permit information due to lack of complete topographic data. The peak flood stage for subbasin 4 is for the storm runoff detention area, not for the subbasin itself. EXTRAN's storage tank option was required for the 25 and 100-year storm events in the Caloosa Estates to account for the effect of submerged conditions during peak flood hours due to high tailwater in C-18. The reduction in outflow rate from those subbasins, due to high tailwater conditions, increased the duration of flooding. Generally, the further upstream the subbasin is, the longer the duration of flooding.

The peak flood stage in subbasin 18 (see Table 9) is much lower than flood stages of adjacent subbasins. This is because the topographic conditions in this subbasin are the lowest, and inflows are limited by surrounding higher spoil banks and levees. In addition, the existing outfall system connecting to the C-18 canal does not have a riser board in place. The outfall system will be changed under the Loxahatchee River Restoration Plan. The water level in this subbasin will be raised to prevent the area from overdrainage and the flood stage will be changed accordingly.

As discussed earlier in the section on present water management in the C-18 basin, there are several existing risers that do not have any boards in place. This allows free exchange of runoff between

TABLE 9. MAXIMUM FLOOD STAGE AND DISCHARGE FOR EACH SUBBASIN UNDER SELECTED DESIGN STORMS

	10-Yr Design Storm		25-Yr Design Storm		100-Yr Design Storm	
	Peak Flood Stage	Peak Discharge	Peak Flood Stage	Peak Discharge	Peak Flood Stage	Peak Discharge
Discharge Subbasin	Ft NGVD	cfs	Ft NGVD	cfs	Ft NGVD	cfs
1	-	78.3	-	102.0	-	146.0
2***	24.0	145.0	24.0	150.0	24.1	169.0
3	22.4	33.3	22.5	51.0	22.7	89.0
4	21.8	80.6	22.8	80.6	22.5	80.6
4*	26.7	68.0	26.8	74.0	27.0	79.0
5	21.4	18.2	21.8	29.3	22.2	49.6
6	21.0	14.7	21.2	22.4	21.5	37.2
7**	20.4	480.0	20.6	500.0	20.8	515.0
8	20.1	75.0	20.2	84.0	20.4	37.0
9	20.6	58.0	20.9	65.6	21.3	91.5
10	18.8	286.0	19.1	337.0	19.4	286.0
11	17.8	22.0	18.0	22.0	18.3	22.0
12A	18.4	128.0	18.6	148.0	18.9	184.0
13	18.7	366.0	18.8	407.0	18.9	473.0
14	17.8	68.0	17.9	83.0	18.2	117.0
15	17.1	232.4	17.3	290.0	17.5	412.0
16	18.1	19.0	18.2	23.0	18.5	32.0
17	16.8	136.4	17.7	152.0	18.0	173.0
18	14.9	379.4	15.3	384.0	16.2	620.6
19	16.8	106.7	17.7	130.0	18.0	352.0
20	15.9	109.0	16.1	109.0	16.5	109.0
21	16.8	74.5	17.9	112.0	18.5	137.0

*Storage area.
**For Caloosa only, the flood stage in the off-site area would be much higher.
***Storage volume based on permit information due to incomplete topographic data.

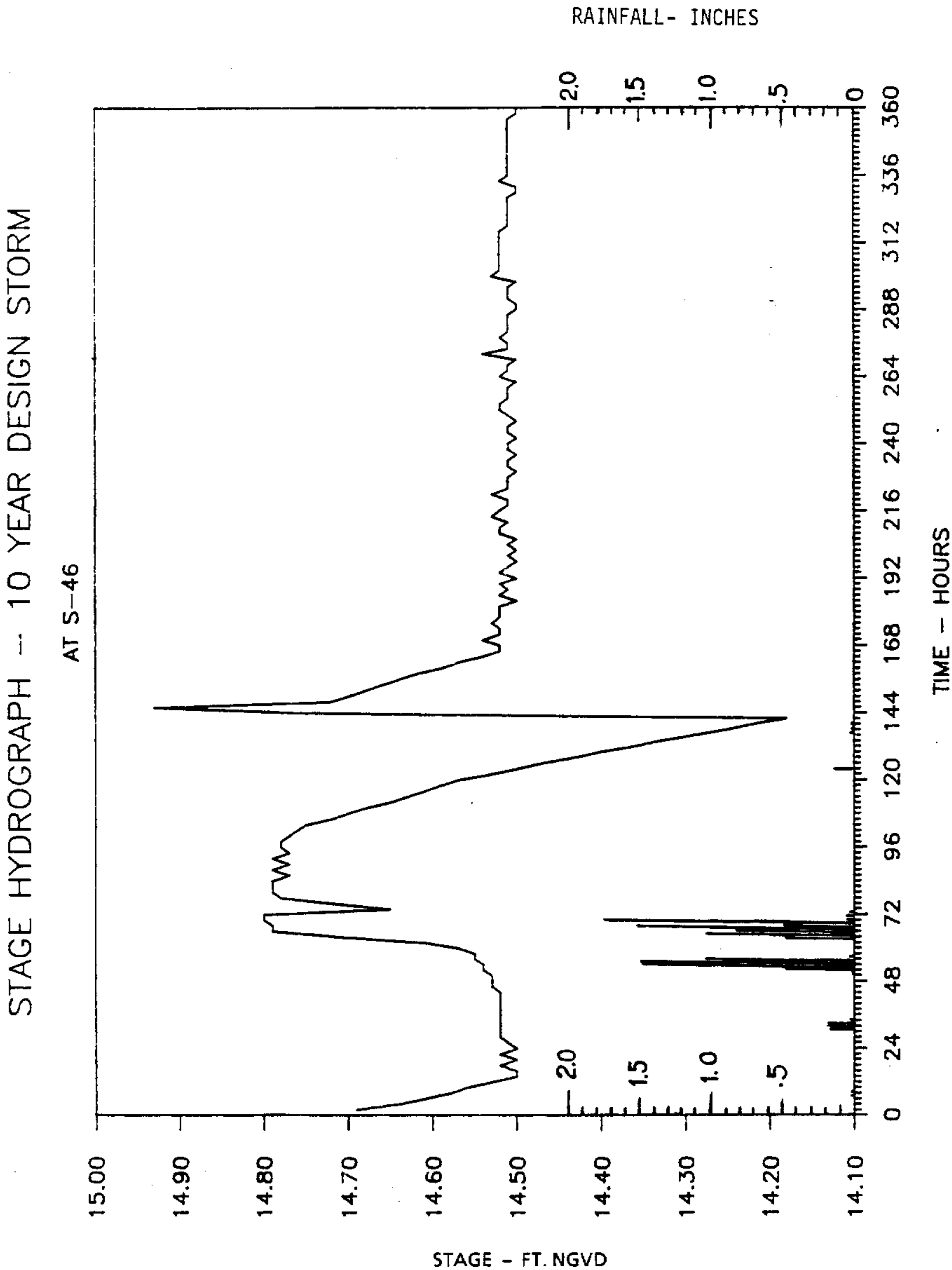

FIGURE 13. Stage Hydrograph Under 10 Year Design Storm at S-46

subbasins and C-18 during severe storm conditions. Figure 14 presents the discharge hydrograph for the outfall structure of subbasin 18 under 10 year design storm. The discharge fluctuated back and forth depending on the water level conditions in the subbasin and C-18 (Figure 13). When the water level became higher, the discharge flowed back into the subbasin until the water level receded. Similar situations occurred in subbasins 15, 17, 19 and 21 since no boards were placed in their risers. Therefore, the subbasins acted as a temporary stormwater storage area for the C-18 basin. Alteration of these outfall structures would have an important effect on the performance of the C-18 basin. If these outfall systems are properly managed, they can be beneficial to the ecosystem of the Loxahatchee Slough, enhance the water quality, and provide additional water supply to the area concerned. In the meantime, adequate flood protection to the area can be provided. Appendix C presents the subbasin discharge hydrographs which resulted from a 10-year design storm.

Table 10 presents the peak runoff rate from each subbasin discharged into C-18 during peak flood hours as compared to the allowable discharge under permit by the District. In summary, the total allowable discharge to C-18 under permit is 4397 cfs (and 5007 cfs if allowable discharges from the proposed water management areas are included) compared to the simulated inflows of 2448 cfs, 2694 cfs, and 3343 cfs for 10-25-, and 100-year design storms, respectively. Due to the routing attenuation in the C-18 canal system, the computed discharge at S-46 for 10-, 25-, and 100-year design storms are 2050 cfs, 2280 cfs, and 2714 cfs, respectively. (Table 7). Therefore, the discharge contributed into C-18 is generally much less than the permitted values, especially for the area upstream of the C-18 weir.

D. Discharge Limitations

The western half of the C-18 west branch is undersized and lacks the capacity to pass the 10-year discharge event analyzed in this study. The studies done by the Corps in 1956 and 1981 assumed no flow from subbasin 1 during peak hours. The present study, however, did not verify that assumption.

The District has found it necessary to restrict runoff rates on permit applications to one inch per 24 hours for the watershed area west of the Beeline Highway (State Road 710). Except for subbasin 4, calculated runoff rates presented in Table 10 for the existing watersheds upstream of the C-18 weir are much less than one inch per 24 hours. The discharge at the C-18 weir was much less during the peak stage downstream of the C-18 weir due to submerged conditions. The peak flow from the area upstream of the C-18 weir was reached after the downstream stage had subsided (see Figures 10 through 12). The discharges at that time were 204 cfs, 289 cfs, and 413 cfs. If these values were used as criteria for runoff allocation from this part of the basin, then the average runoff rate would be 0.18 inch/day, 0.25 inch/day, and

TABLE 10. COMPARISON OF PERMITTED DISCHARGE* VS. PEAK DISCHARGE FROM EACH SUBBASIN DURING PEAK FLOOD HOURS IN THE C-18 BASIN

Subbasin	Permitted** Discharge	10-Year Storm		25-Year Storm		100-Year Storm	
No.	cfs	cfs	in/day	cfs	in/day	cfs	in/day
1	118.0	33.2	0.06	32.0	0.06	46.0	0.08
2	757.0	145.0	0.23	150.0	0.47	169.0	0.53
3	91.0	16.6	0.21	38.0	0.47	72.1	0.89
4	80.6	68.0	0.85	74.0	0.93	79.0	0.99
5	34.0	12.2	0.35	26.0	0.75	46.8	1.36
6	45.0	7.7	0.16	17.0	0.36	31.4	0.66
7	927.0	480.0	1.21	551.0	1.39	515.0	1.30
8	83.0	75.0	6.54	84.0	7.32	37.0	3.22
9	142.8	73.7	2.90	44.4	1.75	17.7	0.70
10	381.5	165.0	1.19	164.0	1.19	277.0	2.00
11	22.2	22.2	1.51	22.2	1.51	22.2	1.51
12	419.5	128.0	0.35	148.0	0.40	184.0	0.50
13	342.0	366.0	3.59	407.0	3.99	473.0	4.64
14	0.0	63.0	1.56	83.0	2.06	117.0	2.90
15	308.8	219.0	1.66	287.0	2.17	412.0	3.12
16	0.0	19.0	0.31	23.0	0.37	32.0	0.52
17	177.0	127.0	2.65	139.0	2.90	147.2	3.07
18	40.0	186.0	2.15	180.0	2.08	247.0	2.85
19	220.2	82.5	0.92	104.0	1.16	224.0	2.49
20	111.0	109.0	5.13	109.0	5.13	109.0	5.13
21	96.3	50.0	3.81	31.0	2.36	26.2	2.00

* Based on permit or conceptual permit information and assume no additional discharge from proposed water management areas.
** Not all permitted discharge can be achieved due to higher tailwater conditions in the C-18

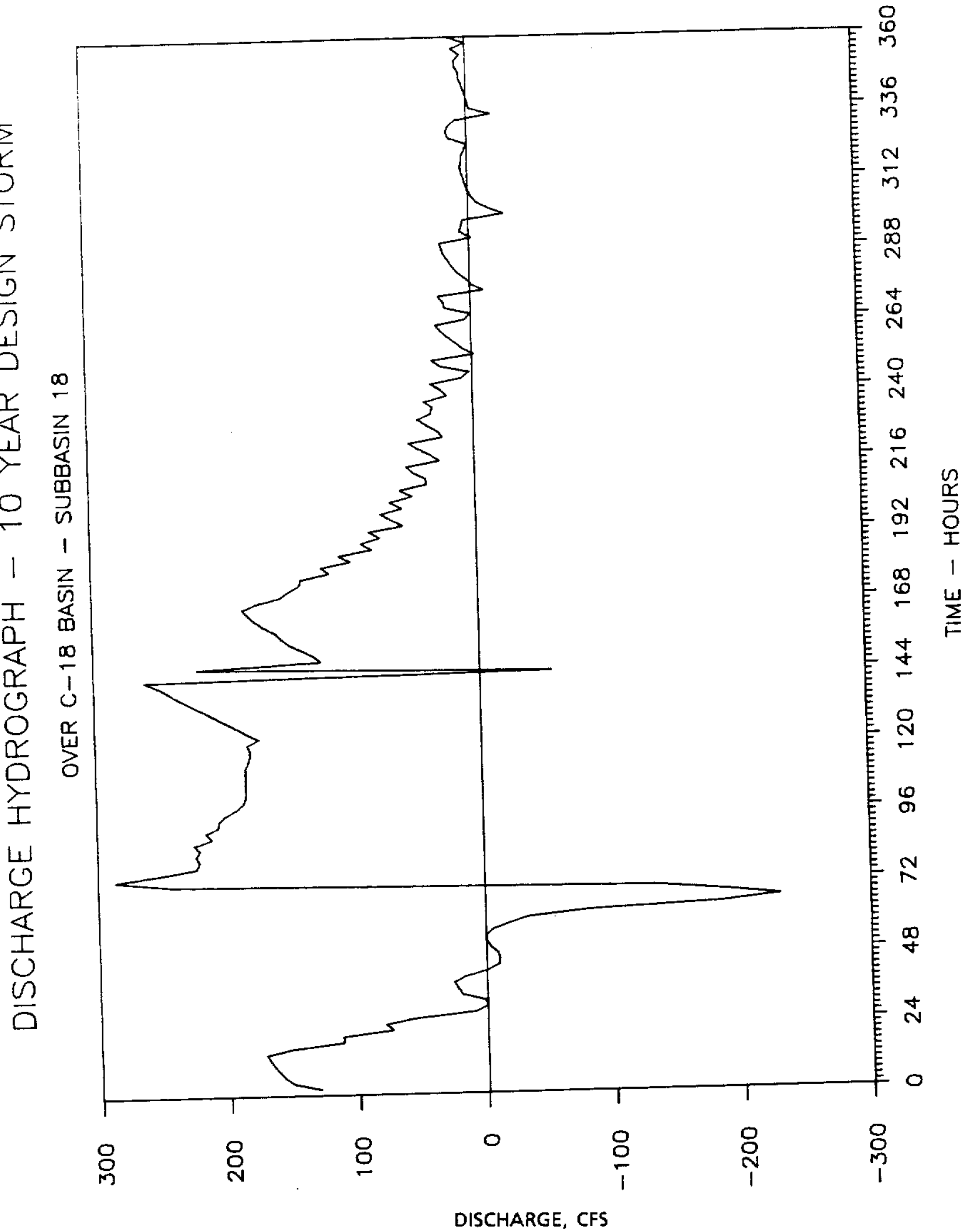

FIGURE 14 Discharge Hydrograph Under 10 Year Design Storm over C-18 Basin at Subbasin 18

0.37 inch/day for 10-, 25-, and 100-year design storm conditions. The 0.25 inch/day is far less than the current criteria of 1 inch/day for the 25-year storm. A storm water detention area (approximately 300 acres) is available for storing pumped runoff from subbasin 4 prior to discharging into C-18. A storm water management system similar to that of subbasin 4 would be beneficial for these basins as far as flood protection and water conservation required for these watersheds. However, the outfall to C-18 would still be restricted due to limited existing canal capacity.

The results of this study show the existing channel capacity in the east branch and main channel of C-18 to be adequate to handle 25 to 100-year design storm events for existing conditions and runoff. Most of the simulated subbasin discharge rates exceed one inch per 24 hours. They range from .35 to 6.5 inches per day for a 10-year design storm (Table 10). The simulated discharge rate for subbasin 12 is only 0.35 inch per 24 hours over the entire subbasin 12 which is due to the fact that there is no positive outfall system for subbasin 12B. Subbasin 12A is the only area that has direct access to C-18 for subbasin 12. The outflow rate for subbasins 14 and 16 are also much less than the allowable rate due to the existing installed stop log riser on the 72 inch CMP culverts with an elevation of 16.5 to 16.90 ft NGVD. Any future improvements to increase the discharge of this outfall system will have an impact on the flow characteristics of the local C-18 reach. This impact will have to be evaluated and addressed when the future improvement plans are available.

E. Degree of Protection

The results of this study, as presented in Table 7, indicate the western half of the west branch of C-18 is under-designed. Under existing runoff conditions the east branch and the eastern half of the west branch are adequately designed to pass the 25-year design storm discharge; however, the computed backwater profile is slightly higher than the original design. The main canal is also capable of handling the 100-year design discharge but with a higher backwater profile than the original design (Table 8).

The following items are presented here for floodplain management consideration:

1. Calculated discharge rates from several subbasins are far below their runoff allocations due to high tailwater conditions which resulted from limited existing channel capacity. As a result, a higher degree of storm water retention would be required. However, in the existing permitted areas, house pad elevations are above the computed flood stages.

2. Runoff from subbasin 12B does not discharge into C-18 due to the lack of a positive outfall system. This area has been allocated with 1 inch per day discharge capacity (i.e. 350 cfs). This additional inflow to the east branch of C-18 would exceed the design capacity of the reach.

3. If all subbasins were to discharge the allowable rate permitted by the District, the capacity of the entire system would be severely exceeded. For example, the existing design capacity for the reach upstream of the C-18 weir is 190 cfs, the permitted discharge for Pratt & Whitney (subbasin 2) alone is 757 cfs.

4. The runoff from the area bounded by State Roads 706, 711, and 710 (called Palmar Estates), located within the off-site area of Caloosa Estates, does not contribute to the Caloosa drainage system during peak hours. This runoff is detained on site and does not reach C-18 until much later. Eventually the runoff from this area is released to C-18 through the Caloosa system when gravity head becomes available.

5. There are some existing shoals in C-18 as presented in Figure 5. There are also some channel constrictions in the reach approximately 800 feet downstream of C-18 weir to the bend where C-18 turns eastward, and the reach near the upstream end of the east branch. Improvement to these shoals and constricted areas will provide a much better degree of flood protection to the existing system.

6. The outflow rate from subbasins 15, 16, 17, 18, and 19 are slightly below the allowable rate by permit; however, these subbasins are currently acting as temporary stormwater storage areas for the basin during severe storm events. On the other hand, the lack of water control on these subbasins has overdrained the area during dry months. Several previous studies indicated that there is an urgent need to restore these wetlands. The rise of the water level in these subbasins for environmental enhancement faces the loss of temporary stormwater storage in the basin and possible backwater to the existing developed area such as the East Pointe subdivision. A system similar to the PGA National Resort can be used to resolve the drainage and environmental enhancement of these subbasins; however, any increasing outflow discharge from these subbasins may adversely impact the east branch and main canal of C-18 due to their limited capacity.

V. CONCLUSIONS AND RECOMMENDATIONS

A. Conclusions

Design storm discharge rates for the current, largely undeveloped C-18 basin are presented in Table 7. The computed discharge at S-46 for the simulated 25-year storm event was 2280 cfs, which is 1140 cfs less than the 3420 cfs structure design discharge capacity.

The sum of the permitted project runoff rates from all subbasins greatly exceeds the design capacity of the C-18 system. Most of these permitted projects are currently either undeveloped or only partially developed. If the unpermitted subbasins were permitted at the existing allowable discharge criteria, the problems would increase.

In order to equitably allocate the remaining C-18 conveyance capacity (assumed to be 1140 cfs) a recommended procedure for redistributing the design discharge for the C-18 basin is presented in the following steps with reference to Table 11:,

1. The model simulated design storm discharges for each subbasin were entered in Column A.

2. Permitted discharge rates for the subbasins were entered in Column B. The unpermitted subbasins were assigned an allowable discharge based on the Everglades runoff formula in the form previously applied for permitting in the C-18 basin. These values were also entered in Column B with parentheses.

3. The positive differences between the permitted (or allowable in the case of unpermitted subbasins) and the design storm simulated discharge (Column B - Column A) were entered in Column C.

4. The differences in Column C were divided by the total basin difference (sum of Colum C) and multiplied by the 1140 cfs to determine each subbasin's share of the remaining capacity (see Column D).

5. Each subbasin's calculated share of the remaining system capacity was then added to the subbasin's simulated existing design storm discharge (Column A + Column D) to derive the allowable discharge (Column E). Column F presents the discharge coefficients for each subbasin as derived by dividing Column E by the subbasin area in acres.

This recommended method for deriving the allowable discharge for the C-18 basin is thus based on

TABLE 11. ALLOCTION OF DISCHARGE FOR SUBBASINS LOCATED EAST OF STATE ROAD 710 WITHIN THE C-18 BASIN

Column	A	B***	C	D	E	F
Subbasin Numbe	Computed Discharge cfs	Permitted Discharge cfs	Difference Col B - A cfs	Discharge Adjustment cfs	Redistrib'd Discharge cfs	Discharge Coefficient csm
7	500.0	927.0	427.0	287.4	787.4	54.0
8	84.0	83.0	-1.0	0.0	84.0	198.0
9	44.4	(142.8)	98.4	66.2	110.6	115.5
10	164.0	396.0	232.0	156.2	320.2	62.4
11	22.2	(14.8)	-7.4	0.0	22.2	40.6
12	148.0	(570.7)	422.7	284.5	432.5	31.4
13	407.0	342.0	-65.0	0.0	407.0	107.4
14	83.0	(189.7)	106.7	71.8	154.8	103.6
15	287.0	308.8	21.8	14.7	301.7	61.3
16	23.0	(251.7)	228.7	154.0	177.0	76.5
17	139.0	177.0	38.0	25.6	164.6	91.8
18	180.0	40.0	-140.0	0.0	180.0	55.8
19	104.0	220.2	116.2	78.2	182.2	54.7
20	109.0	111.0	2.0	1.3	110.3	139.0
21	31.0	0.0	0.0	0.0	31.0	63.5*
			SUM** = 1693.5			

csm: cfs per square mile
* Subbasin 21 is within SIRWCD
** Only positive differences were summed
*** Discharge rates shown in parentheses are based on the Everglades runoff formula

the simulated existing discharge rates for design storms. Accordingly, the method is consistent with the criteria in other South Florida basins where post development discharge rates are not to exceed predevelopment discharge rates, except the remaining capacity of the system is also distributed among the subbasins.

Subbasins upstream of the C-18 weir do not have access to the remaining capacity of the C-18 canal system. Accordingly, the allowable discharge for these subbasins should be based on the simulated rates for the design storm under existing conditions as presented in Table 9.

The resulting discharge coefficients for determining allowable discharge rates for individual projects is shown in Figure 15.

B. Recommendations

The following recommendations are proposed to ensure future development within the C-18 basin will be afforded an adequate flood protection without overtaxing the primary outfall system:

1. The present policy of prohibiting pump systems from discharging directly into the C-18 canal should continue.

2. Storage compensation for development within the C-18 floodplain should be based on the 100-year subbasin stages depicted in Figure 16.

3. Road crown and house pad elevations for development within the C-18 basin should at a minimum, be above the applicable return flood stages presented in Table 8 and Figures 16 and 17. Routings for road design should also consider tailwater stages in C-18 resulting from the 10-year design storm.

4. The allowable discharge rates for the respective subbasins should be based on the discharge coefficients proposed in Figure 15. Design discharges for proposed projects should consider the 25-year design tailwater stages in the appropriate segment of the C-18 canal displayed in Table 8.

5. The removal of restrictions to flow downstream of the C-18 weir should be reviewed.

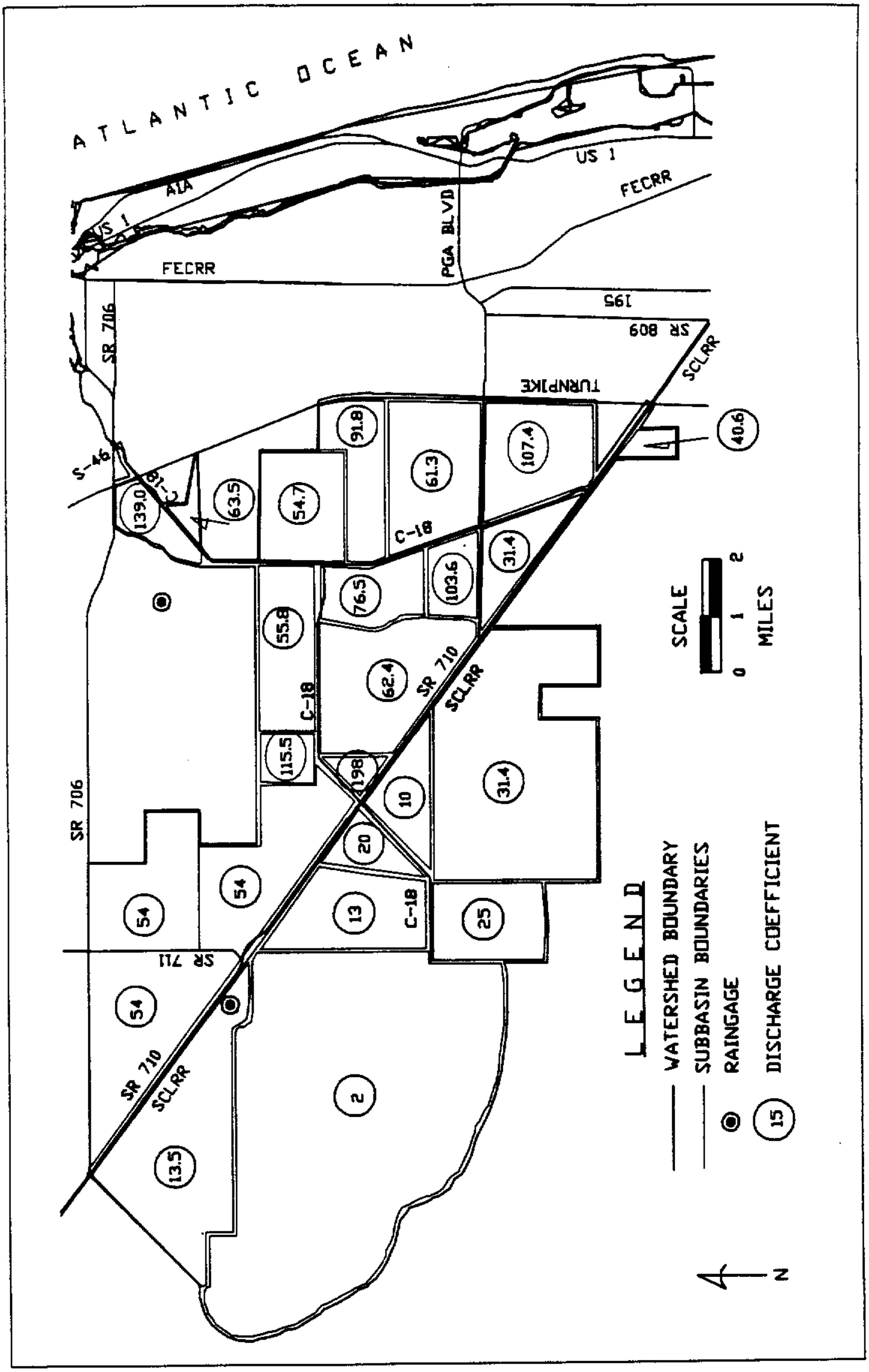

FIGURE 15. Discharge Coefficient,Ce, for New Development. Permitted Discharge Qp = Ce * A/640 Where A is Drainage Area in Acres

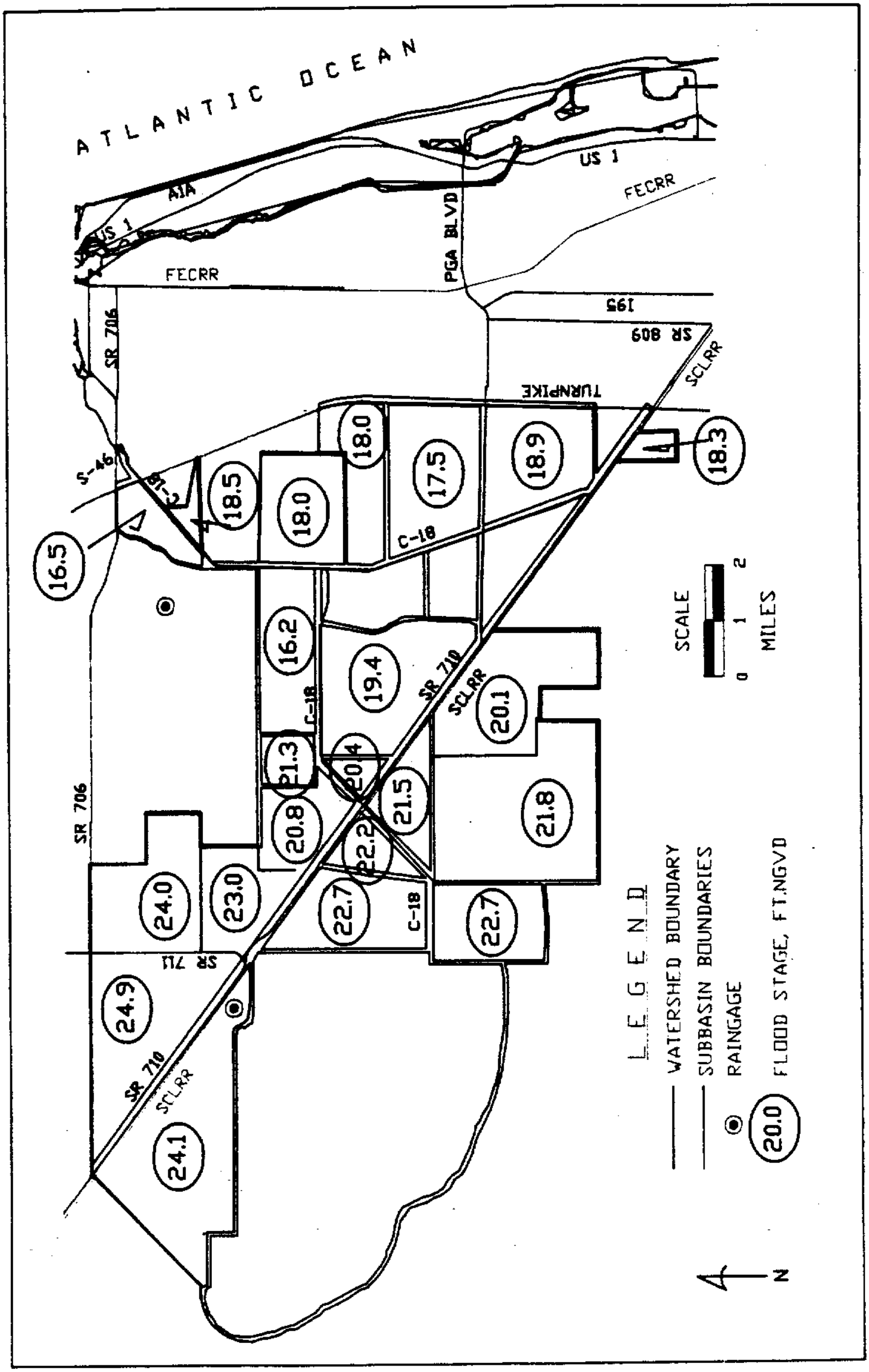

FIGURE 16 Peak Flood Stage (Ft. NGVD) During a 1-in-100 Year Storm Event

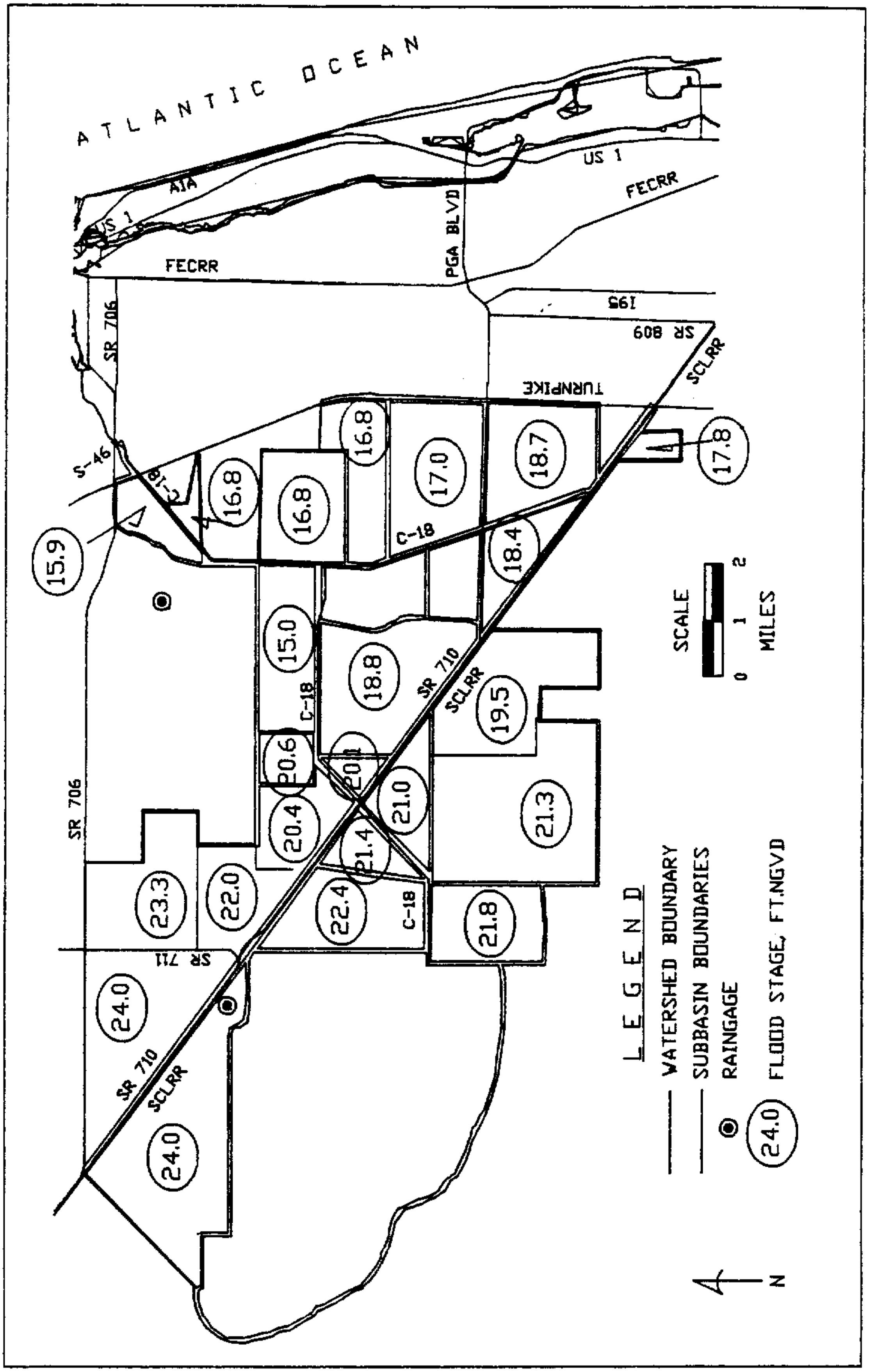

FIGURE 17. Peak Flood Stage (Ft. NGVD) During a 1-in-10 Year Storm Event

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APPENDIX A

APPENDIX A

List of Permitted Areas and Flow Rates

Permittee	Permit No	Average Acres	Permitted Q-cfs
Corbett Wildlife (Subbasin 1)	50-00251-S	13,387.0	118.0
Pratt & Whitney (Subbasin 2)	50-01355-S	6,919.0	757.0
Pratt & Whitney (Subbasin 2)	50-01355-S	201.0	
United Technologies (Subbasin 2)	6P-82-221	11.2	0.8
Indian Trail WCD (Subbasin 3)	50-00136-S	2,240.0	91.0
Citrus Ridge (Subbasin 4)	50-00689-S	1,919.0	80.6
NPBCWCD* Unit 8 (Subbasin 6)	50-00037-S	1,137.0	45.0
Caloosa (Subbasin 7)	50-00474-S	9,291.0	927.0
Foxtrail, Inc. (Subbasin 8)	50-00467-S	243.0	83.0
Foundation Land (Subbasin 10)	50-01626-S	3,268.5	396.0
Foundation Land (Subbasin 12A)	50-01626-S	1,207.0	116.9
NPBCWCD Unit 8 (Subbasin 12B)	50-00037-S	7,588.0	350.0
PGA National (Subbasin 13)	50-00617-S	2,341.0	342.0
Foundation Land (Subbasin 15)	50-01626-S	3,085.0	308.8
Foundation Land (Subbasin 15)	50-01053-S	60.0	38.0
East Pointe S/D (Subbasin 17)	50-00532-S	640.0	148.0
Old Marsh (Subbasin 17)	50-01411-S	446.4	29.0
Foundation Land (Subbasin 18)	50-01626-S	949.2	40.0
Foundation Land (Subbasin 19)	50-01626-S	1,817.6	220.2
DuBois Farms (Subbasin 20)	50-00805-S	388.0	109.0
P. B. Dev. & Sales (Subbasin 20)	50-01322-S	30.8	2.0
TOTAL		**57,169.7	4,202.3

* NPBCWCD represents Northern Palm Beach County Water Control District

** This acreage represents 89.71% of total C-18 basin permitted by the SFWMD

APPENDIX B

MODEL CALIBRATION

APPENDIX B
MODEL CALIBRATION

The September 21-24, 1983 storm event was used to calibrate the model using 1985 land use and water management conditions. The rainfall amounts during this event ranged from 11.32 inches recorded at MRF 231 (South Indian River Water Control District) to 12.55 inches at MRF 54 (Pratt & Whitney). The antecedent rainfall recorded at these two stations are presented in Table B-1.

The hourly rainfall distribution at MRF 231 was used in the calibration run and is presented in Figure B-1 in which the simulation began at 1400 hours, September 21, 1983. The break point stage and discharge data at S-46 (data available from zero hours, September 22, 1983) are presented in Figures B-1 and B-2. The step-like discharge hydrograph at S-46 for this storm event (Figure B-1) may be caused by the fact that the rate of the gate opening is very slow along with the manual operation of the gate. According to the recorded gate operational chart, the manual operation occurred at 8 a.m., September 24, 1983, and the gates were opened to 5 ft for about 2 hours. Then the gates were closed down to 4.15 ft for the next 10 hours, then further closed to 3.65, 3.07, 2.42, and 1.83 ft at a time interval of approximately 10 to 14 hours. The gates were closed and put back on regular automatic setting at 10 a.m., September 26. Peak discharge occurred while the gates were manually opened to 5 ft for 2 hours. The headwater stage at S-46 during this storm period was held at 12.90 to 13.45 feet NGVD, as shown in Figure B-2; therefore, the stage and discharge characteristics in the C-18 canal depend on the gate operations and inflow characteristics from the subbasins.

The simulated flood stage and discharge hydrographs at S-46 are depicted in Figures B-1 and B-2. In general, the stage hydrographs agree with the record values; however, the computed discharges tend to be slightly higher than the historical ones. This difference was attributed to the manual operation of the gates during this storm event.

The EXTRAN model requires input data for initial flow, velocity, and water level conditions to initialize the hydraulic properties of the system. The daily readings at S-46 and the C-18 weir were the only available source of data for the estimation of the initial condition prior to the storm. In general, the results were in agreement for the system downstream of the C-18 weir. Upstream conditions, however, were not adequately simulated. The initial stage upstream of the C-18 weir dropped from 18.20 ft to 17.80 ft NGVD within the first few hours due to the lack of continuing inflow from its tributaries. In other words, the estimation of runoff recession hydrographs are

TABLE B-1. Rainfall Distribution for September 1983

Sept.	MRF 231	MRF 54	Sept.	MRF 231	MRF 54
1	1.29	0.20	16	0.26	1.00
2	0.00	0.00	17	0.05	0.00
3	0.00	0.00	18	0.25	1.40
4	0.00	0.00	19	0.39	0.10
5	0.00	0.00	20	0.00	0.00
6	0.00	0.00	21	0.03	0.00
7	0.00	0.00	22	0.59	2.20
8	0.00	0.00	23	4.50	0.85
9	0.00	0.00	24	6.20	9.50
10	0.02	0.00	25	0.00	0.00
11	0.00	0.00	26	0.12	0.55
12	0.00	0.00	27	0.03	0.00
13	1.75	0.20	28	0.00	0.00
14	0.29	0.40	29	0.00	0.00
15	1.24	0.50	30	0.00	0.00
Totals				17.00	16.90

required for each subbasin in order to adequately simulate the initial hydraulic properties of the C-18 basin upstream of the C-18 weir. This can be observed from the actual simulated stage hydrograph at the C-18 weir which is a slow recession hydrograph, Figure B-3). The computed stages are fairly consistent with the daily readings of 18.60 feet and 18.46 feet NGVD for the September 1983 storm, except at the initial period. In conclusion, the routing model used in this study is considered valid for the flood study in this basin.

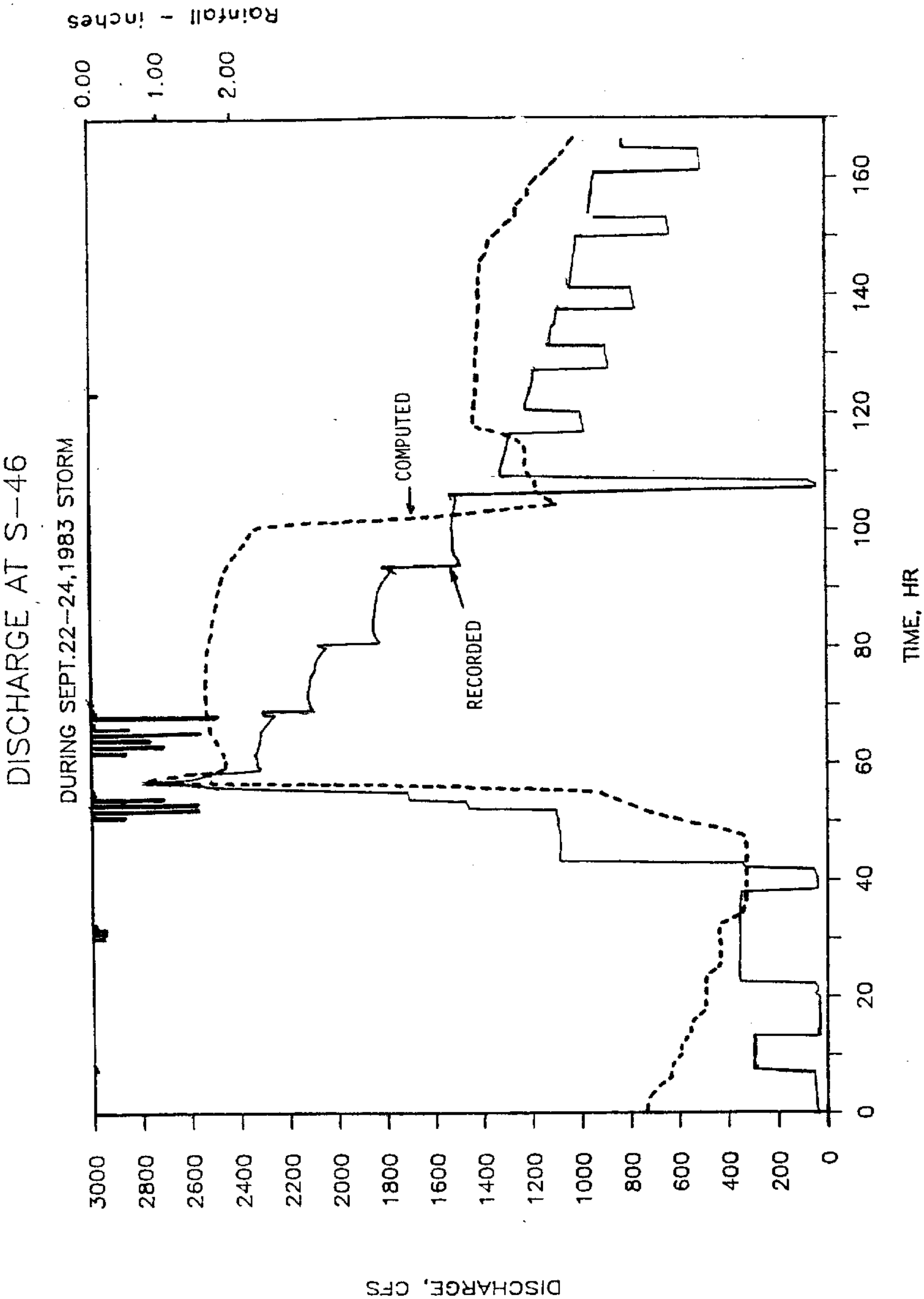


FIGURE B-1. Comparison of Computed and Recorded Discharge at S-46.

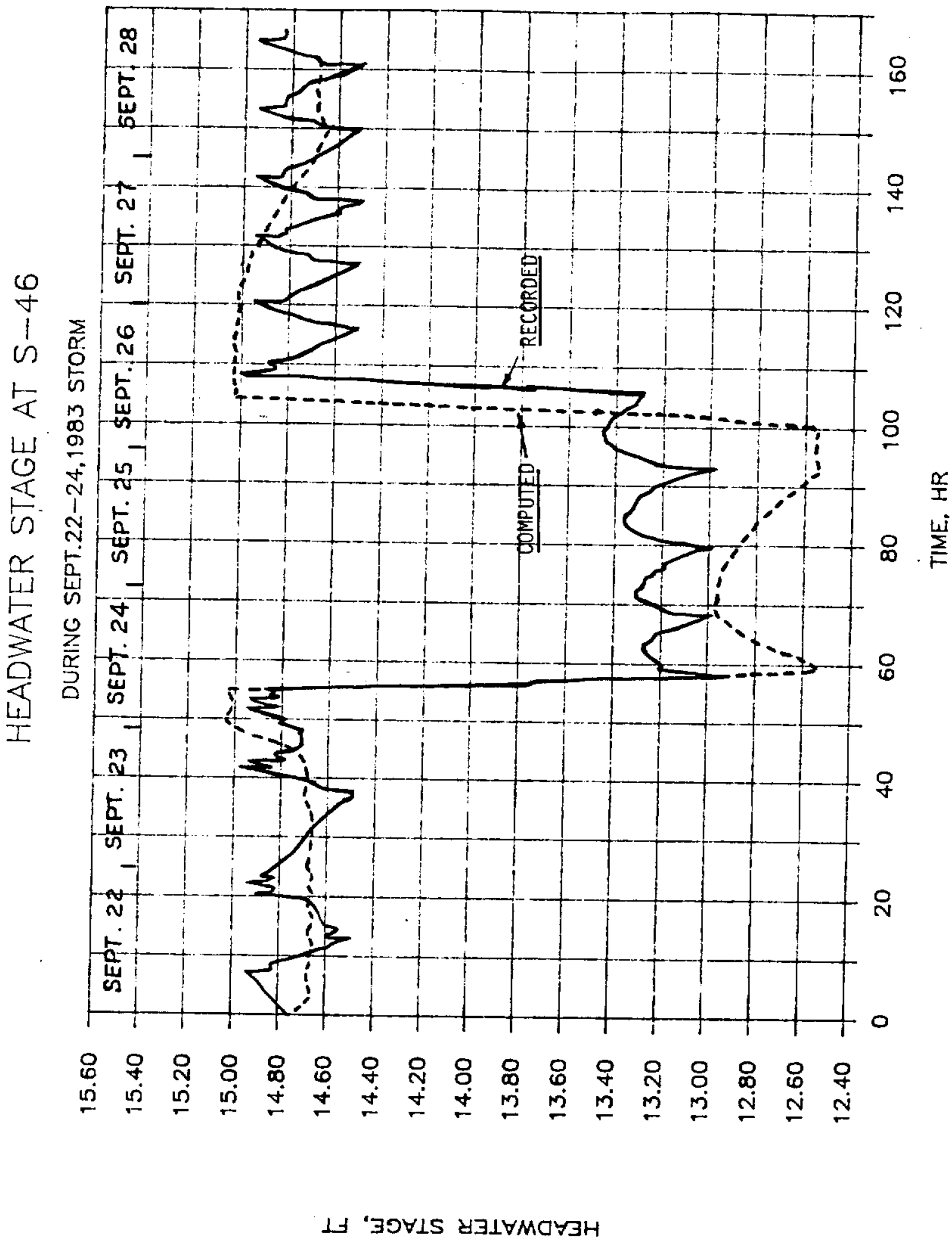


FIGURE B-2. Comparison of Computed and Recorded Headwater Stage at S-46.

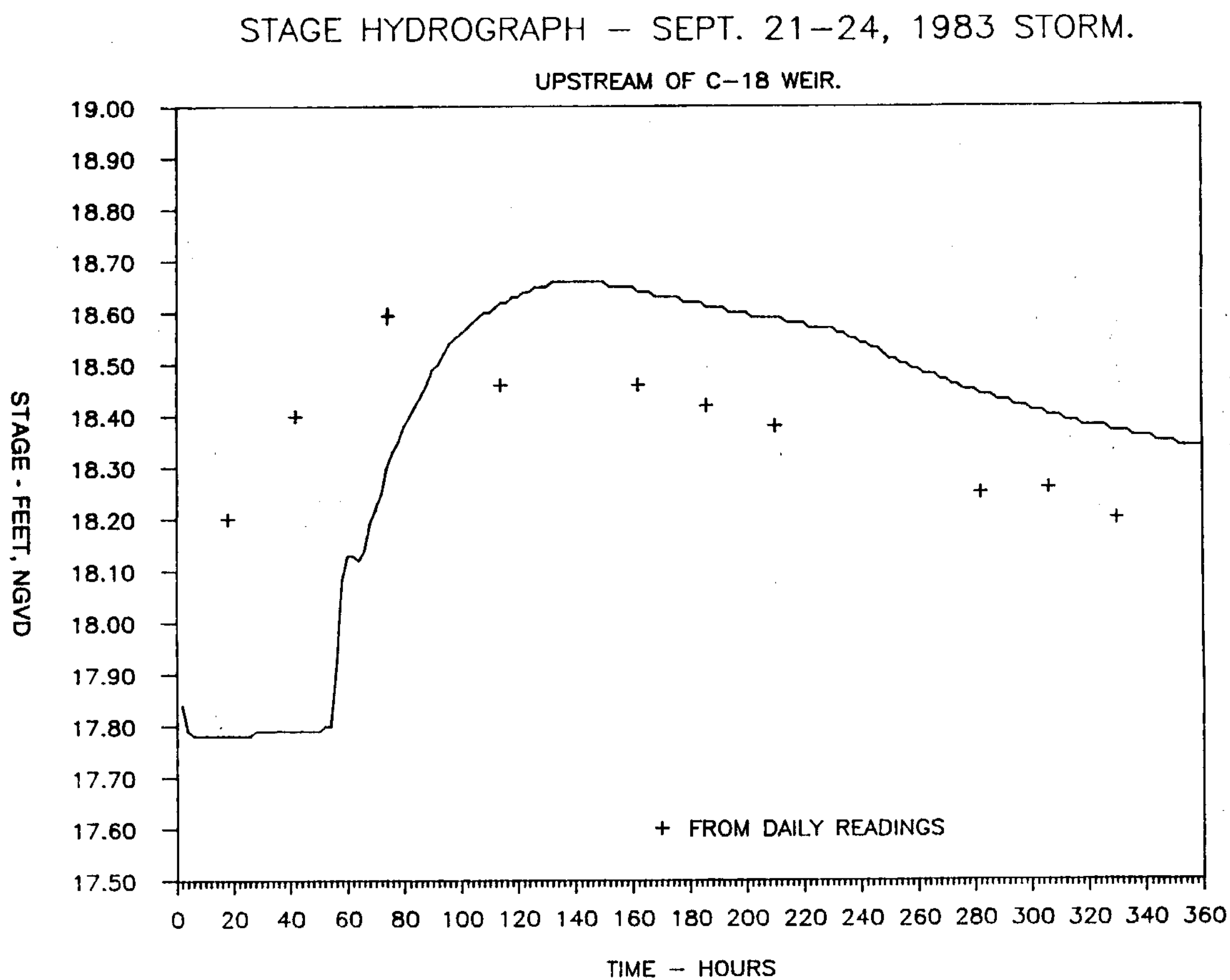


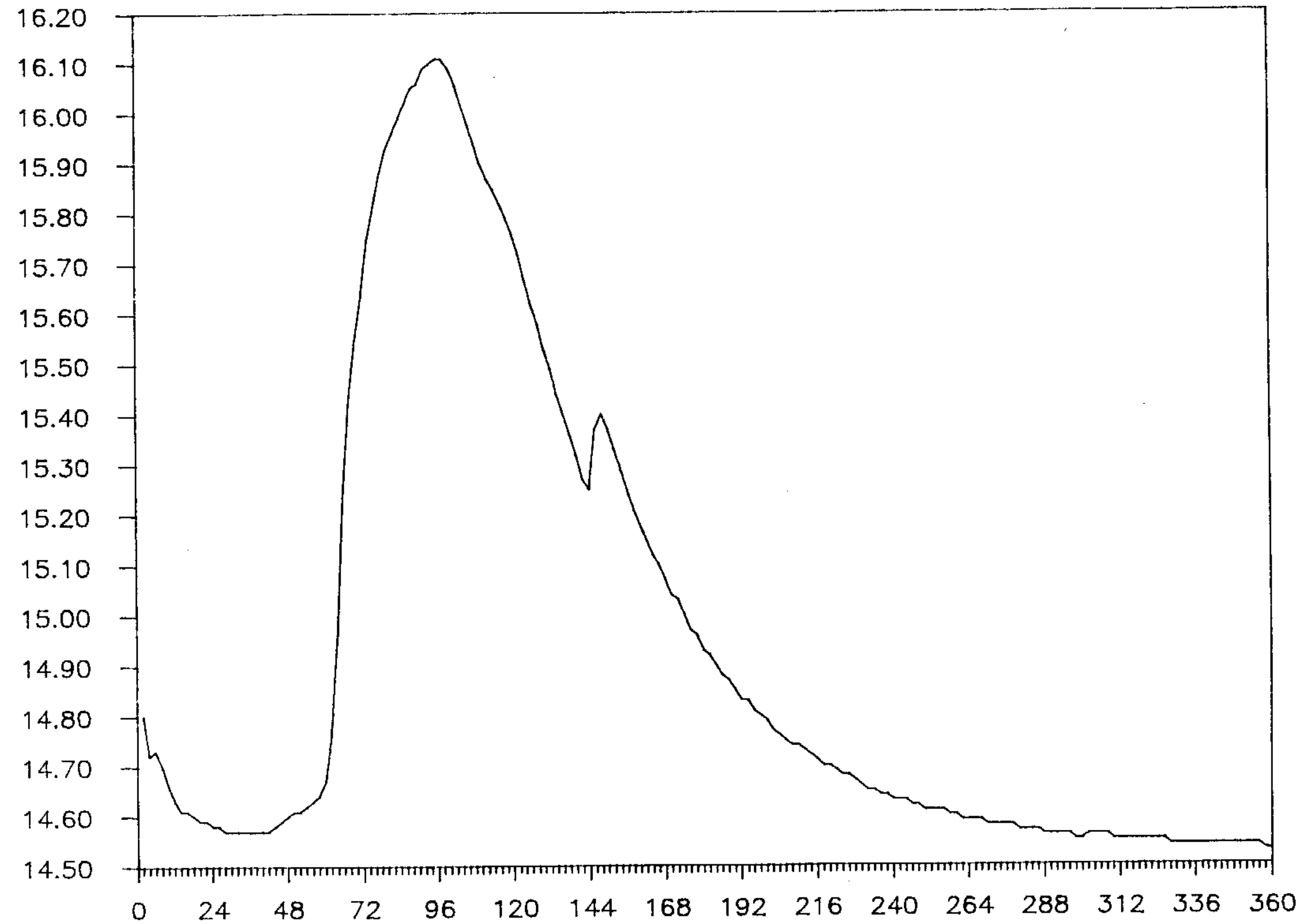
FIGURE B-3 Stage Hydrograph Upstream of the C-18 Weir - Sept. 21-24, 1983 Storm.

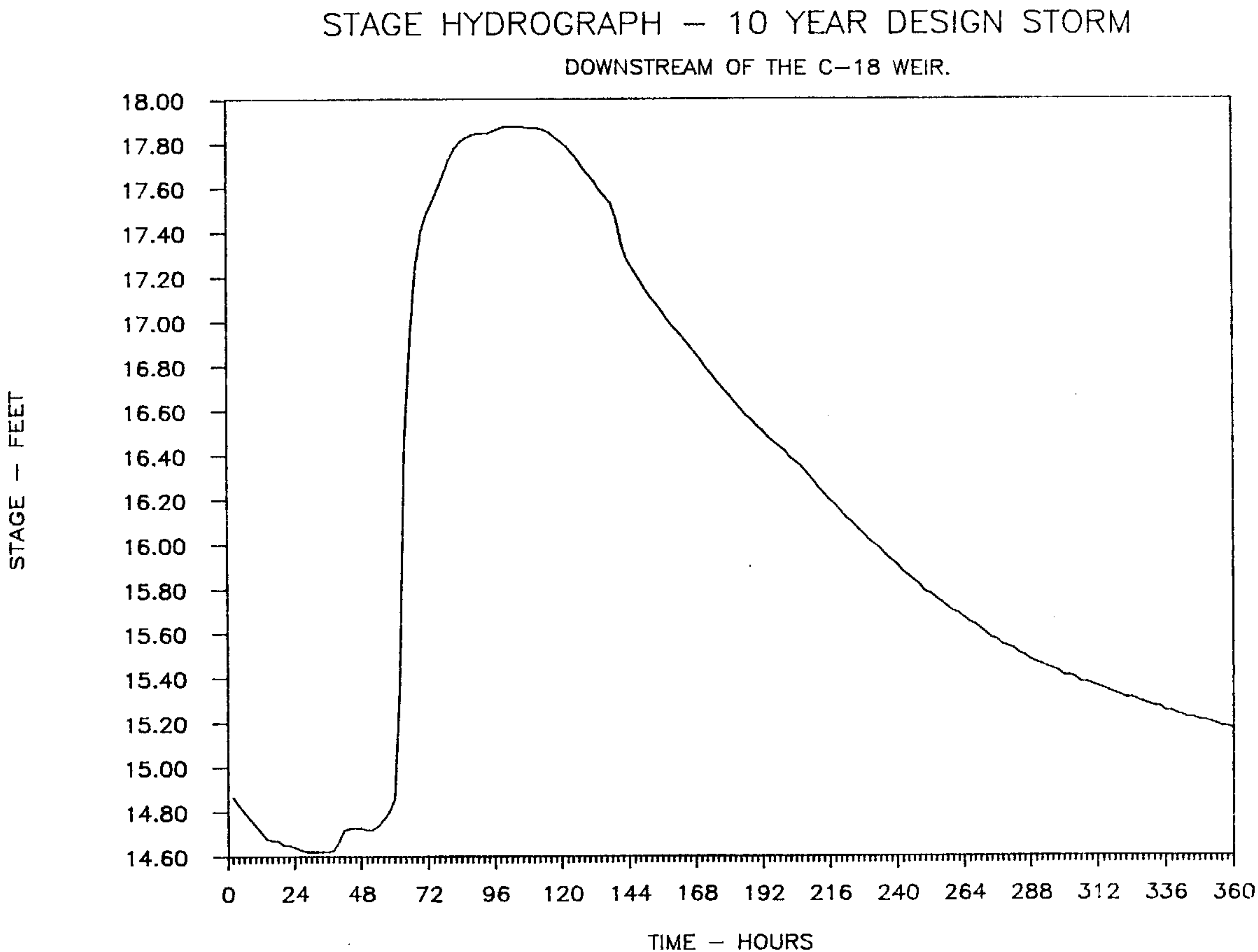
APPENDIX C

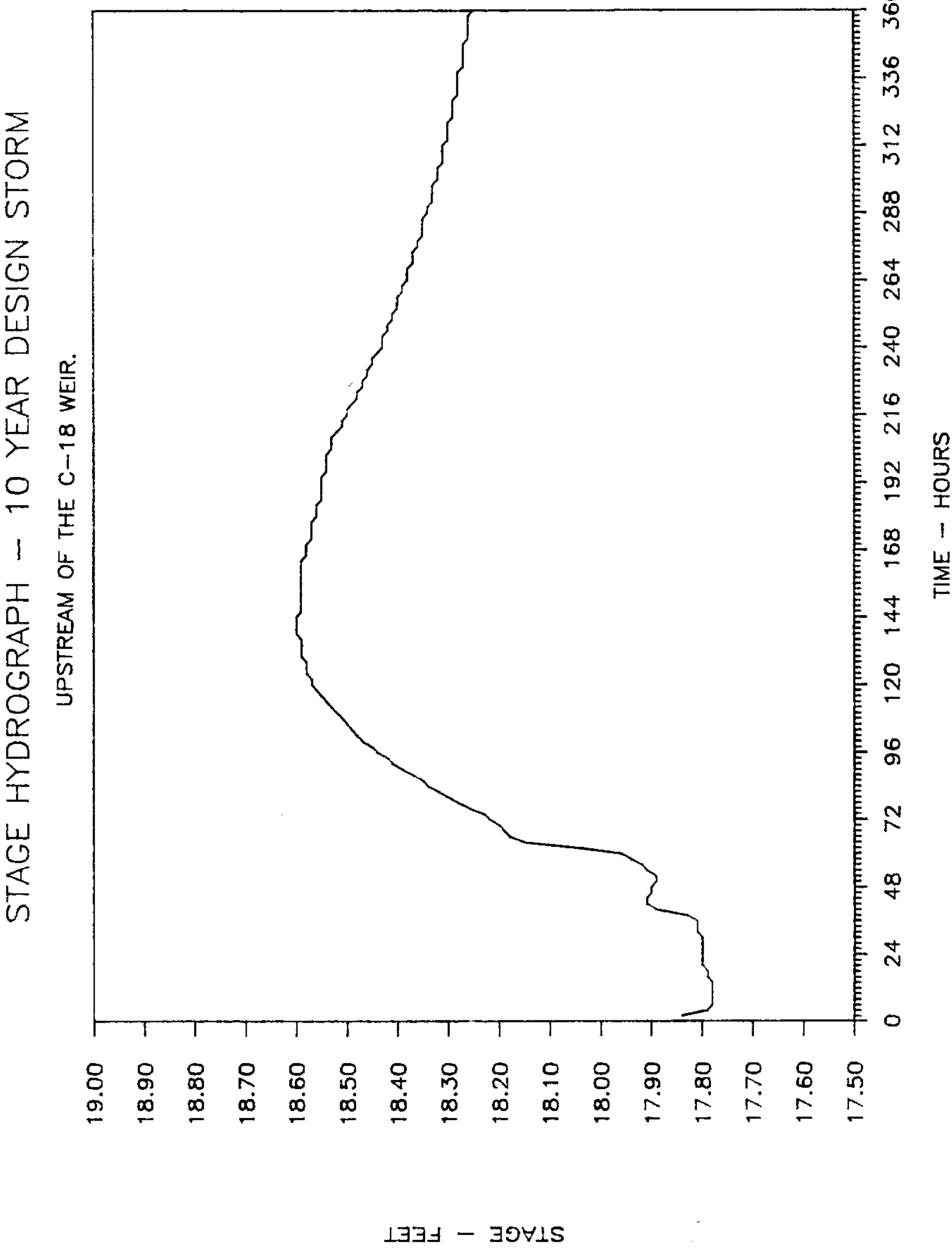
STAGE AND DISCHARGE HYDROGRAPHS

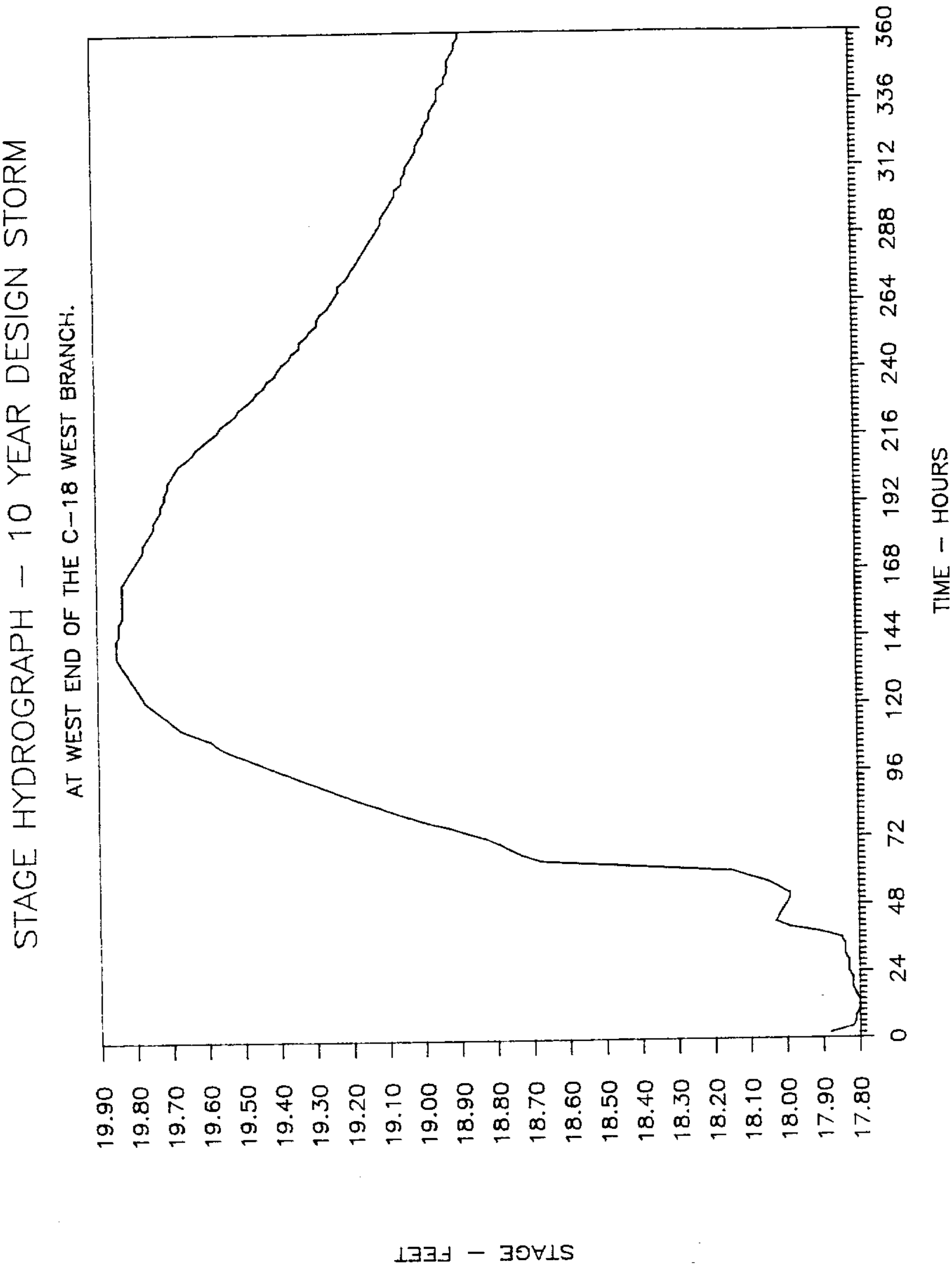
STAGE HYDROGRAPH – 10 YEAR DESIGN STORM

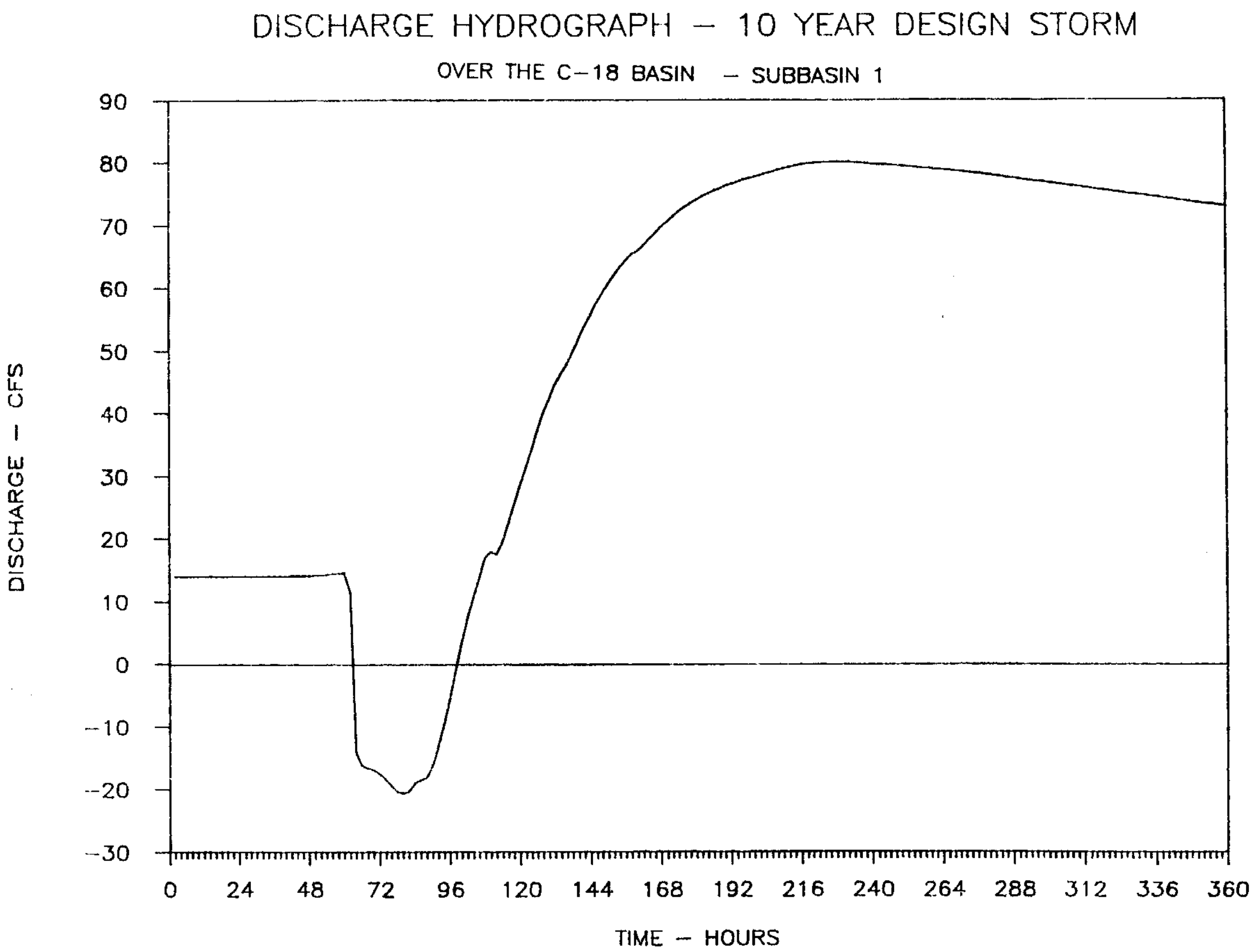
AT SR710 AND EAST BRANCH OF THE C–18 CANAL.




STAGE HYDROGRAPH – 10 YEAR DESIGN STORM
DOWNSTREAM OF THE C–18 WEIR.
18.00
17.80
17.60
17.40
17.20
17.00
16.80
16.60
16.40
16.20
16.00
15.80
15.60
15.40
15.20
15.00
14.80
14.60
STAGE – FEET
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
TIME – HOURS


STAGE HYDROGRAPH — 10 YEAR DESIGN STORM
UPSTREAM OF THE C-18 WEIR.
TIME — HOURS
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
STAGE — FEET
19.00
18.90
18.80
18.70
18.60
18.50
18.40
18.30
18.20
18.10
18.00
17.90
17.80
17.70
17.60
17.50

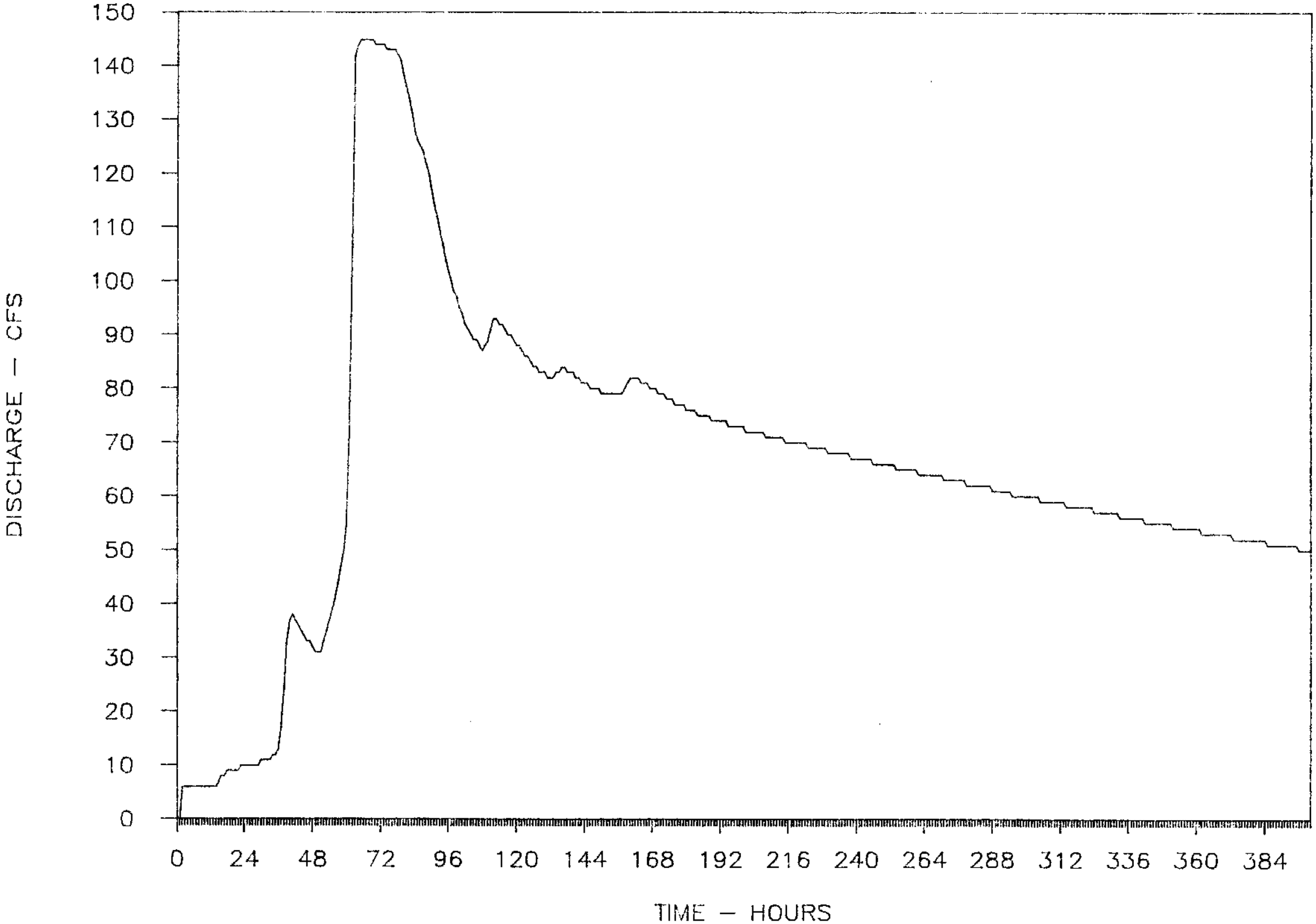

STAGE HYDROGRAPH — 10 YEAR DESIGN STORM
AT WEST END OF THE C-18 WEST BRANCH.
STAGE — FEET
19.90
19.80
19.70
19.60
19.50
19.40
19.30
19.20
19.10
19.00
18.90
18.80
18.70
18.60
18.50
18.40
18.30
18.20
18.10
18.00
17.90
17.80
TIME — HOURS
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360

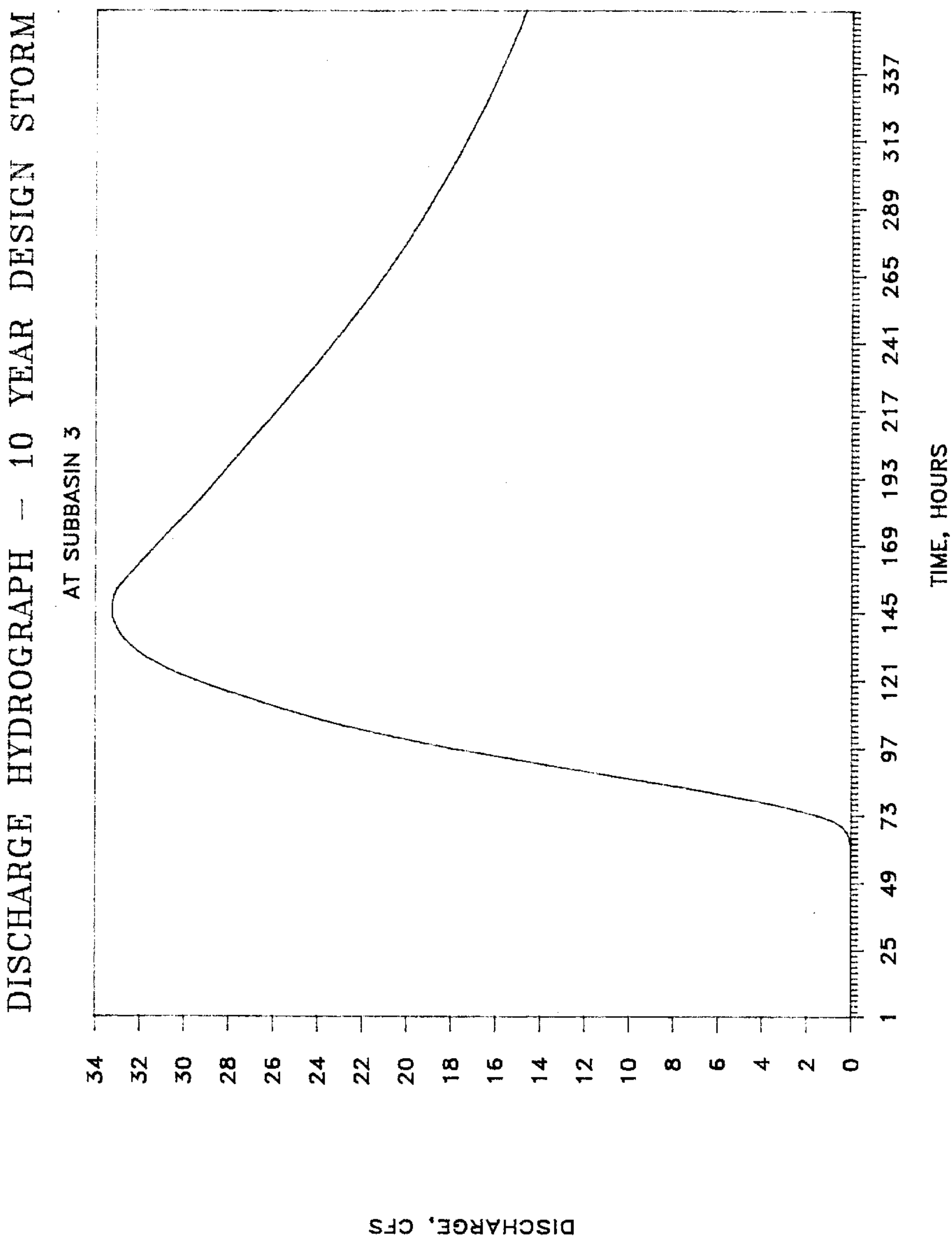


DISCHARGE HYDROGRAPH — 10 YEAR DESIGN STORM
OVER THE C—18 BASIN — SUBBASIN 1
DISCHARGE — CFS
90
80
70
60
50
40
30
20
10
0
—10
—20
—30
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
TIME — HOURS

BASIN DISCHARGE HYDROGRAPH – 10 YEAR DESIGN STORM

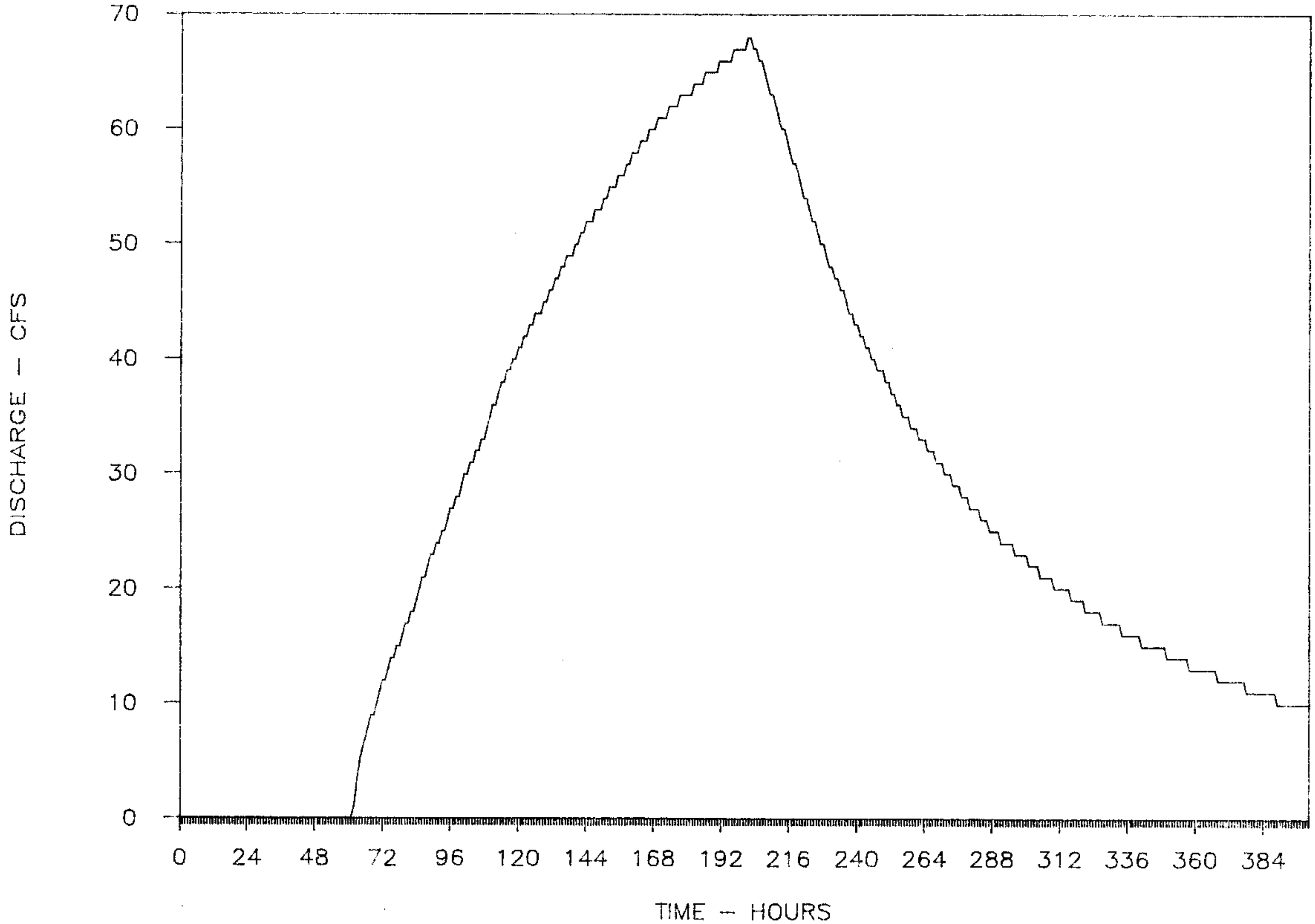
AT SUBBASIN 2




DISCHARGE HYDROGRAPH — 10 YEAR DESIGN STORM
AT SUBBASIN 3
DISCHARGE, CFS
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
TIME, HOURS
1
25
49
73
97
121
145
169
193
217
241
265
289
313
337

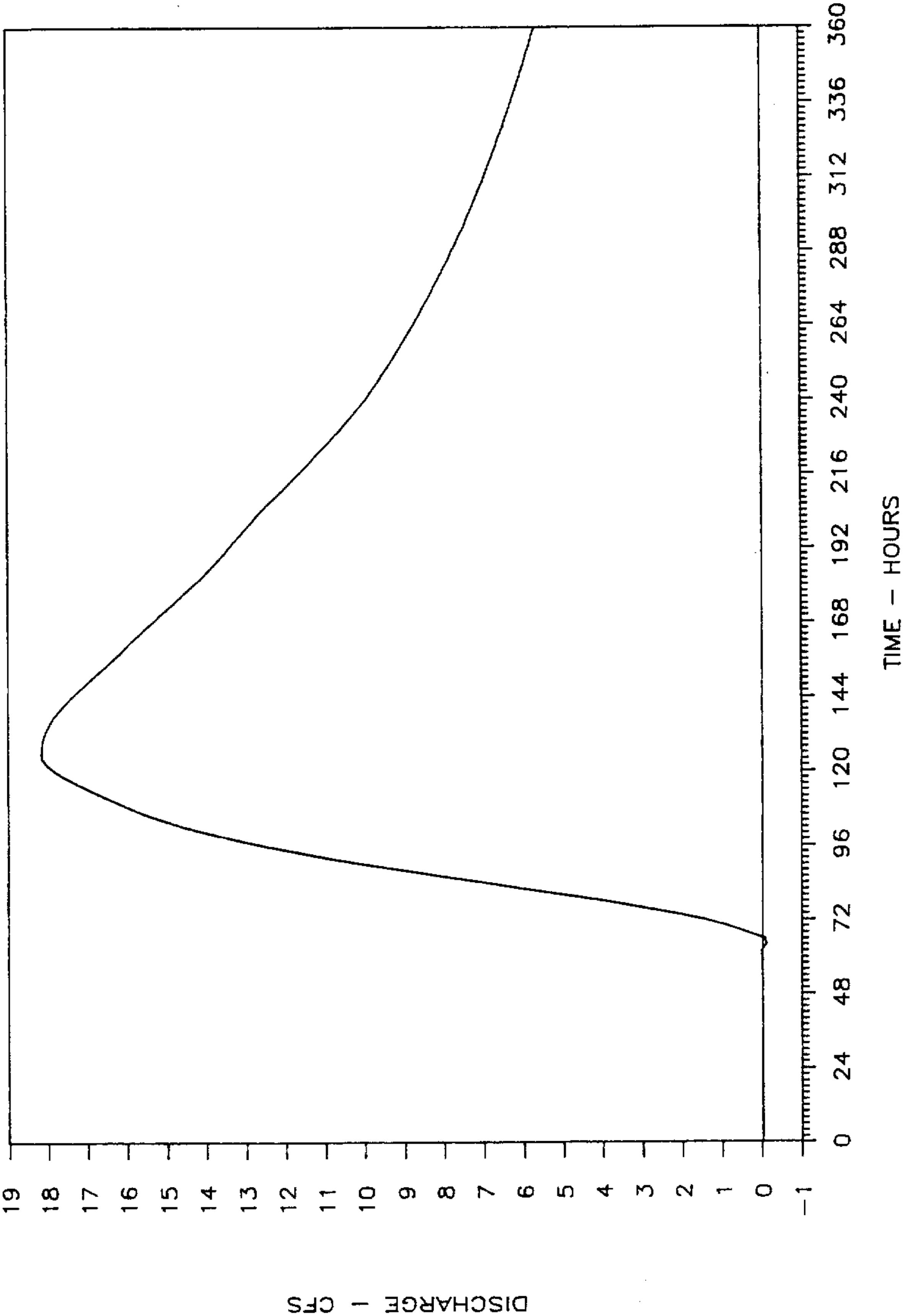
BASIN DISCHARGE HYDROGRAPH — 10 YEAR DESIGN STORM

AT SUBBASIN 4



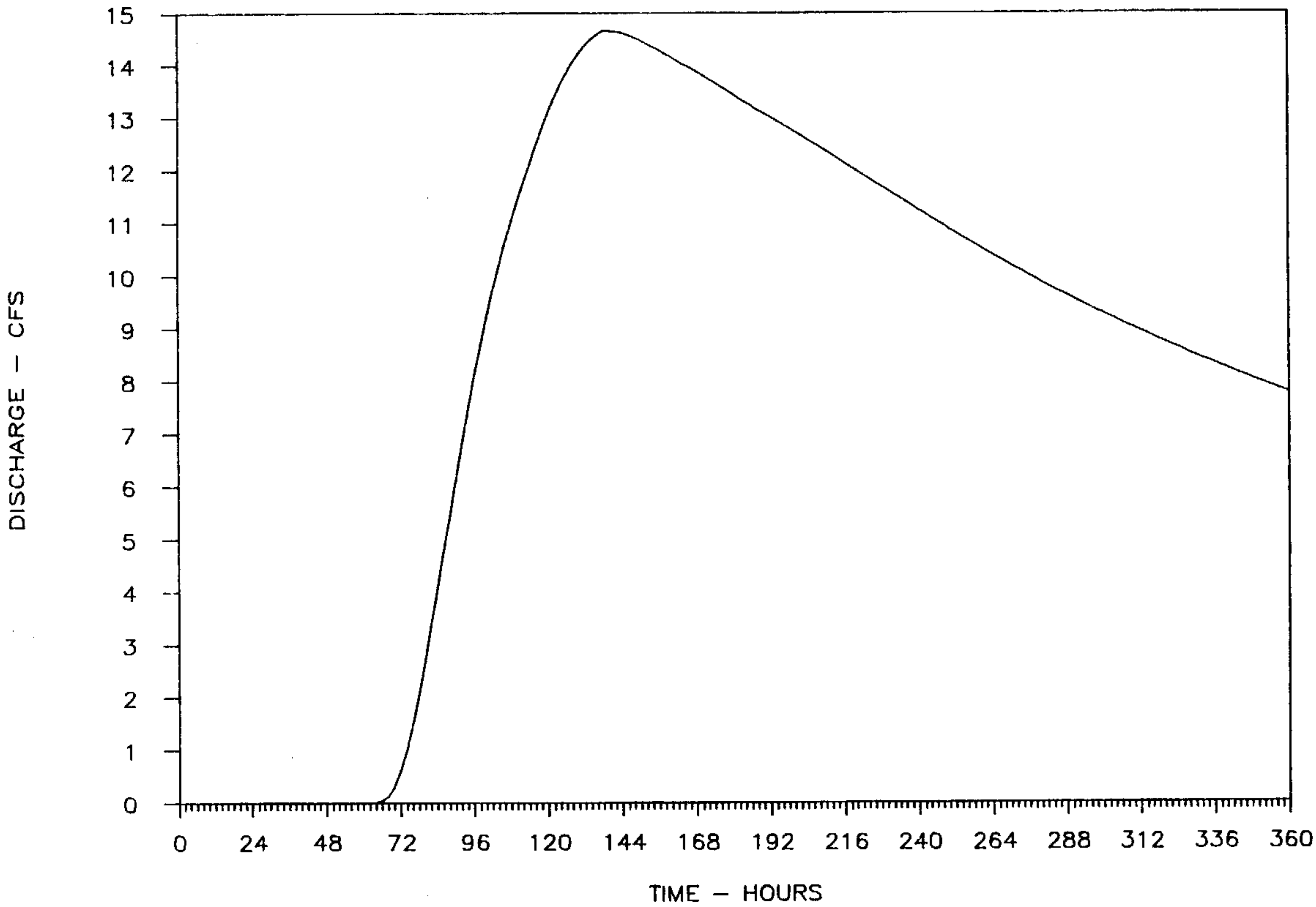
DISCHARGE HYDROGRAPH — 10 YEAR DESIGN STORM

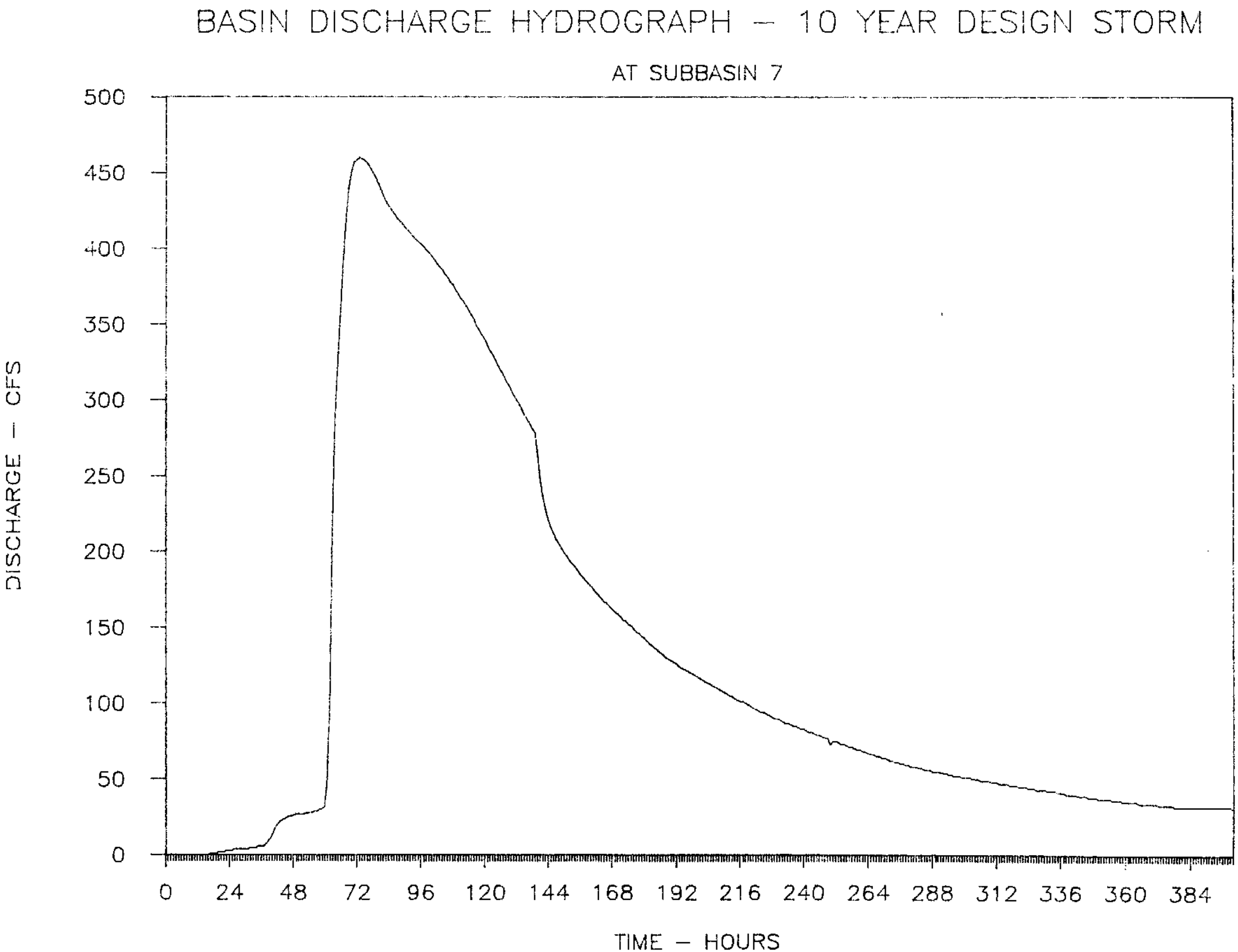
OVER THE C-18 BASIN — SUBBASIN 5

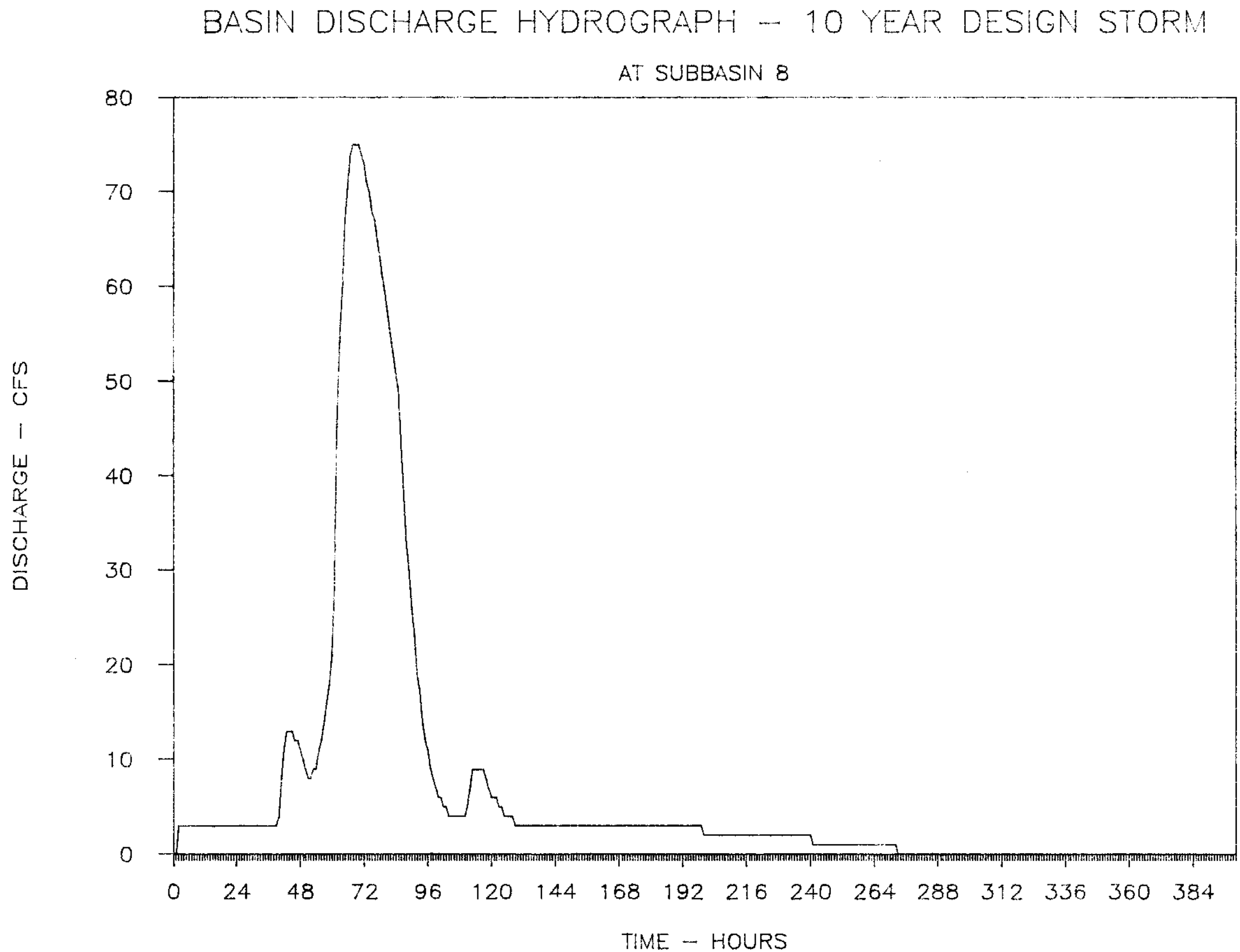


DISCHARGE HYDROGRAPH — 10 YEAR DESIGN STORM

OVER THE C-18 BASIN — SUBBASIN 6

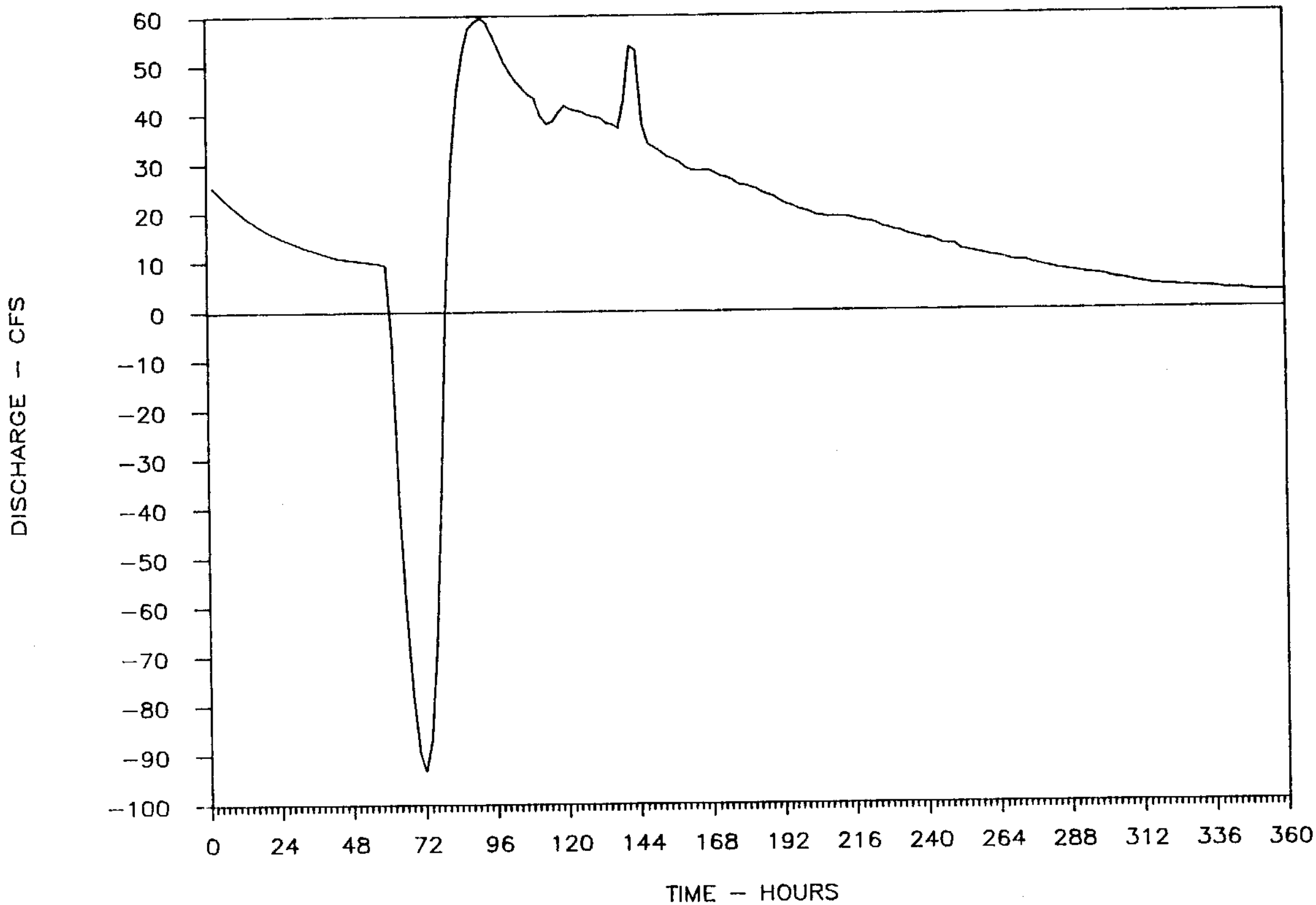



BASIN DISCHARGE HYDROGRAPH — 10 YEAR DESIGN STORM
AT SUBBASIN 7
DISCHARGE — CFS
500
450
400
350
300
250
200
150
100
50
0
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
384
TIME — HOURS


BASIN DISCHARGE HYDROGRAPH — 10 YEAR DESIGN STORM
AT SUBBASIN 8
80
70
60
50
40
30
20
10
0
DISCHARGE — CFS
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
384
TIME — HOURS

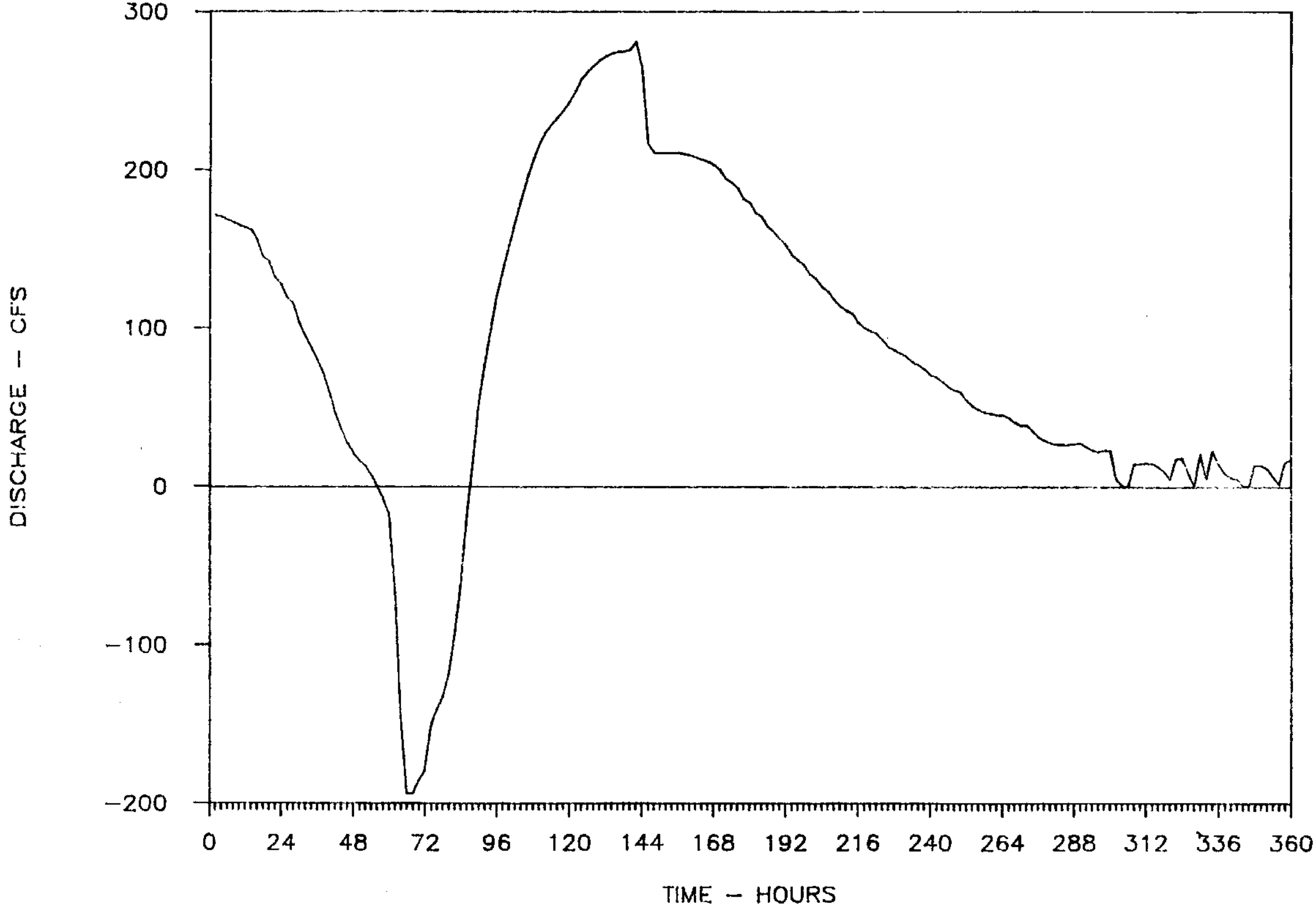
DISCHARGE HYDROGRAPH — 10 YEAR DESIGN STORM

OVER THE C–18 BASIN — SUBBASIN 9



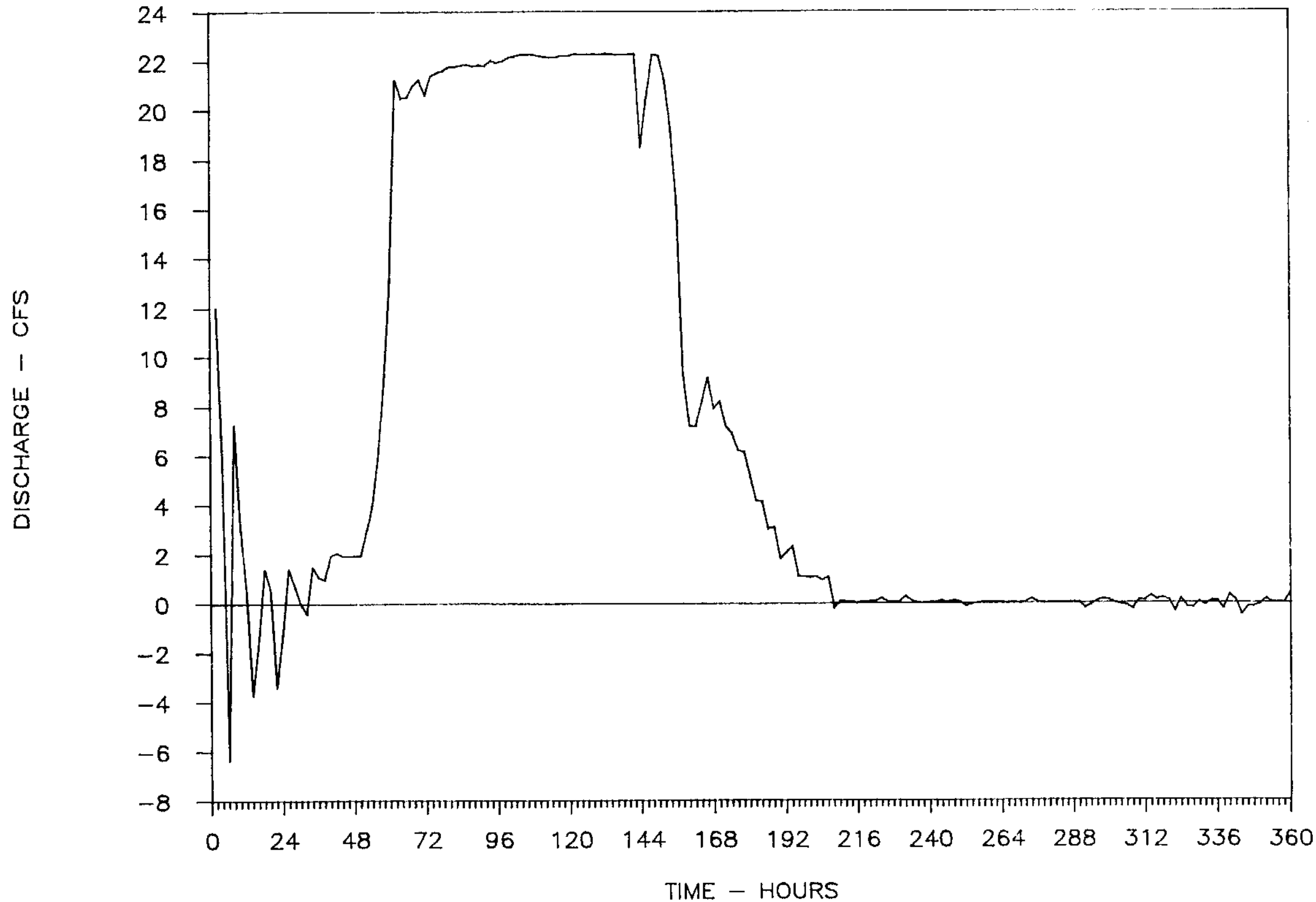
DISCHARGE HYDROGRAPH — 10 YEAR DESIGN STORM

OVER THE C-18 BASIN – SUBBASIN 10



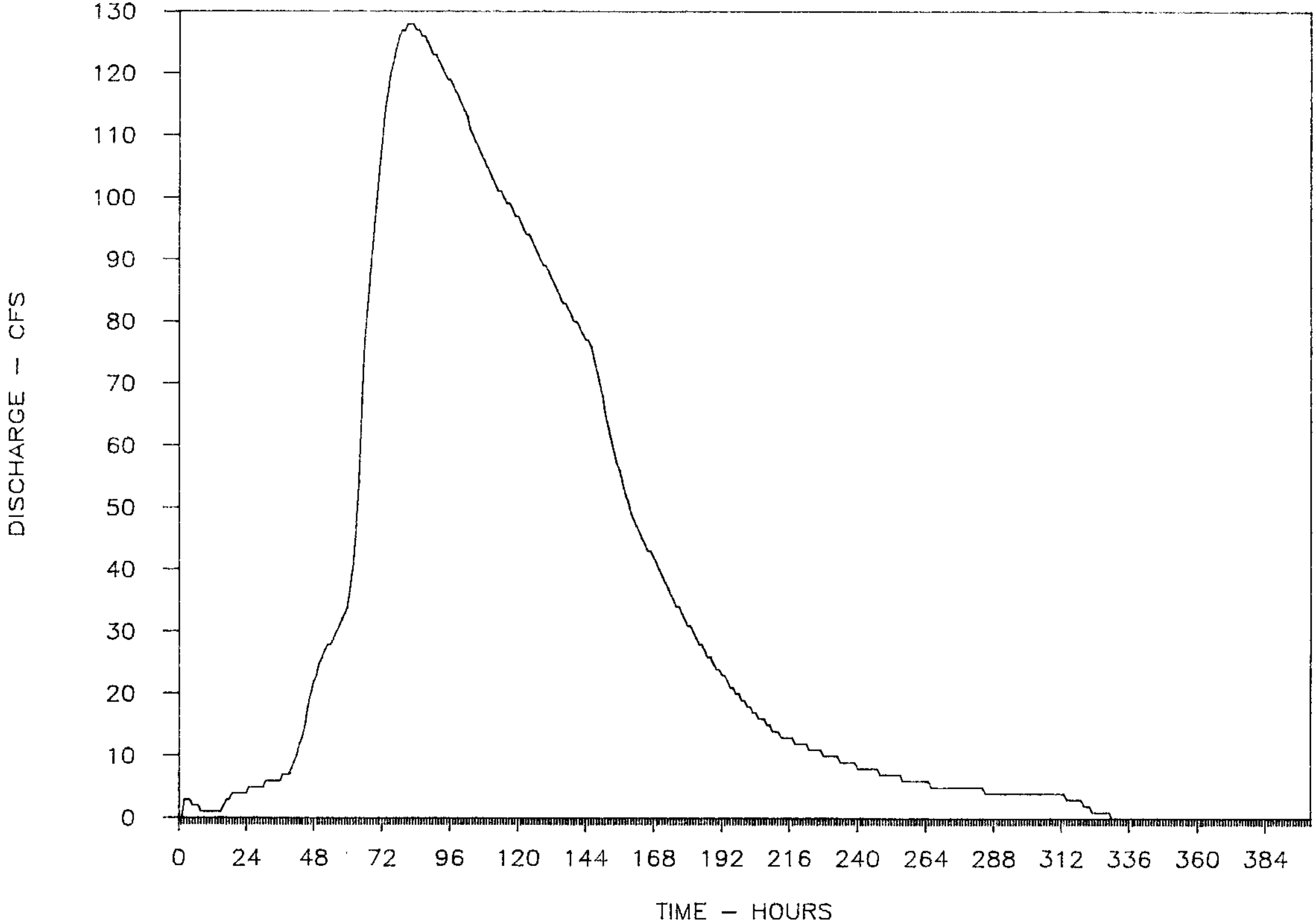
DISCHARGE HYDROGRAPH – 10 YEAR DESIGN STORM

AT SR710 AND EAST BRANCH OF THE C–18 CANAL.



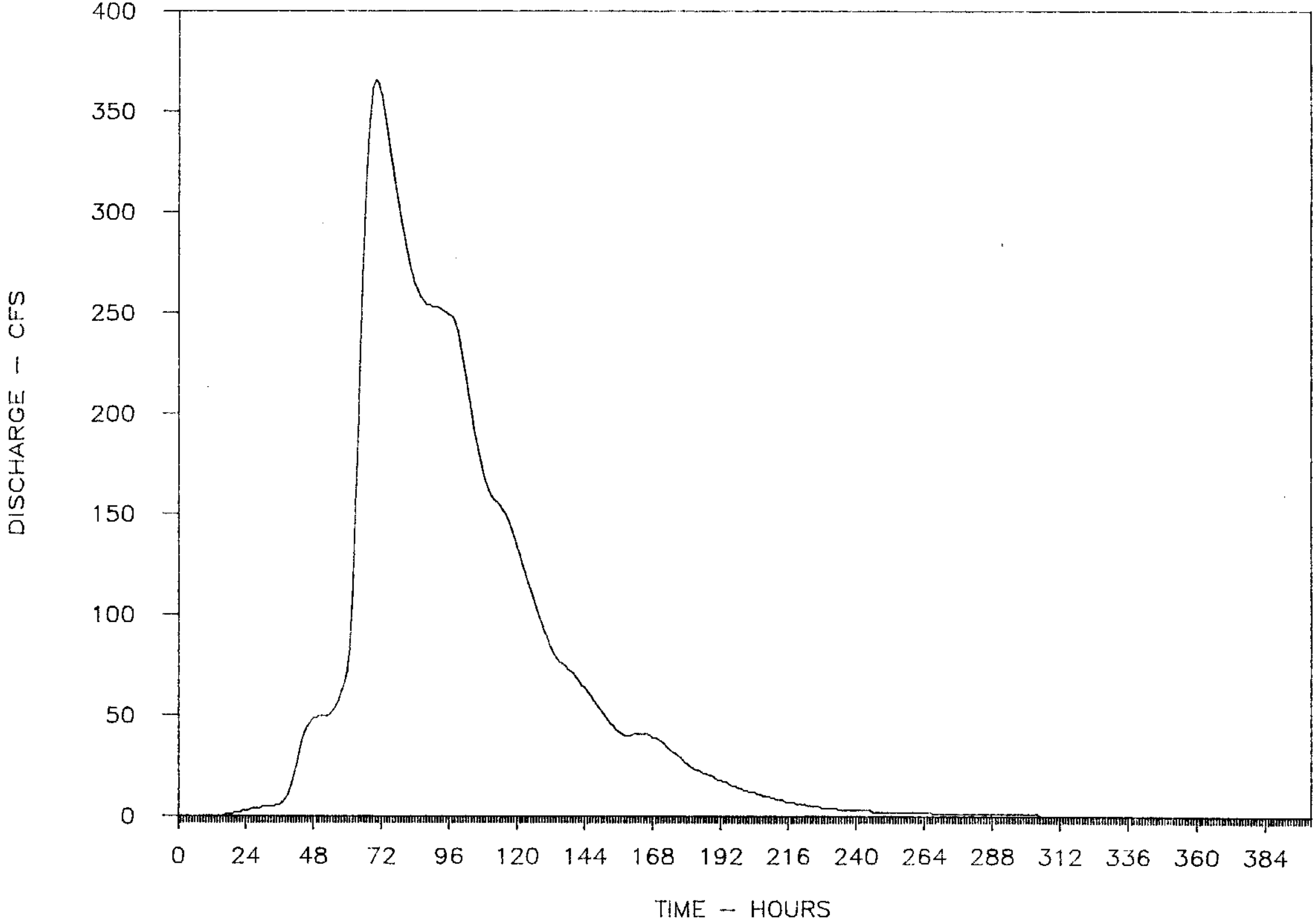
BASIN DISCHARGE HYDROGRAPH – 10 YEAR DESIGN STORM

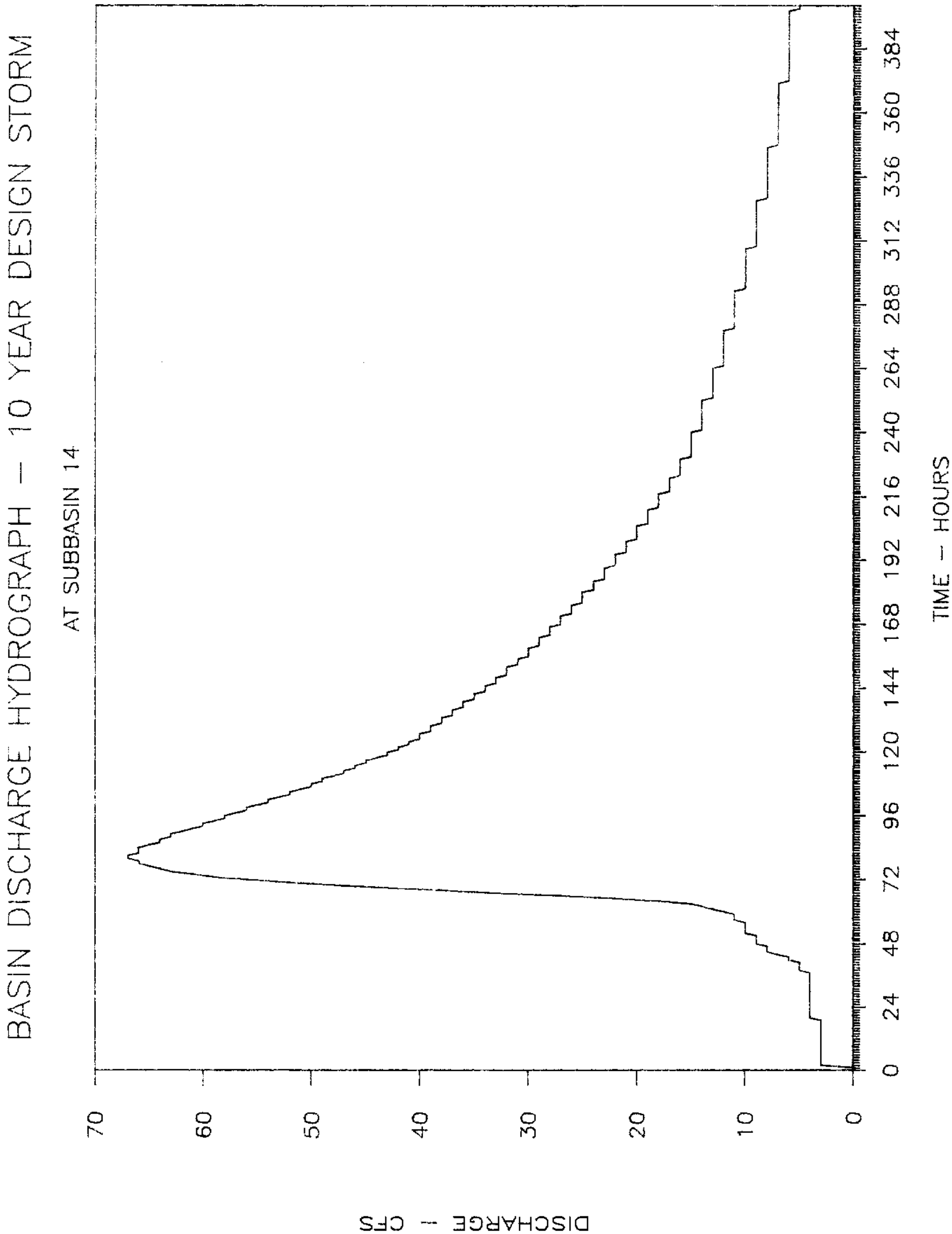
AT SUBBASIN 12

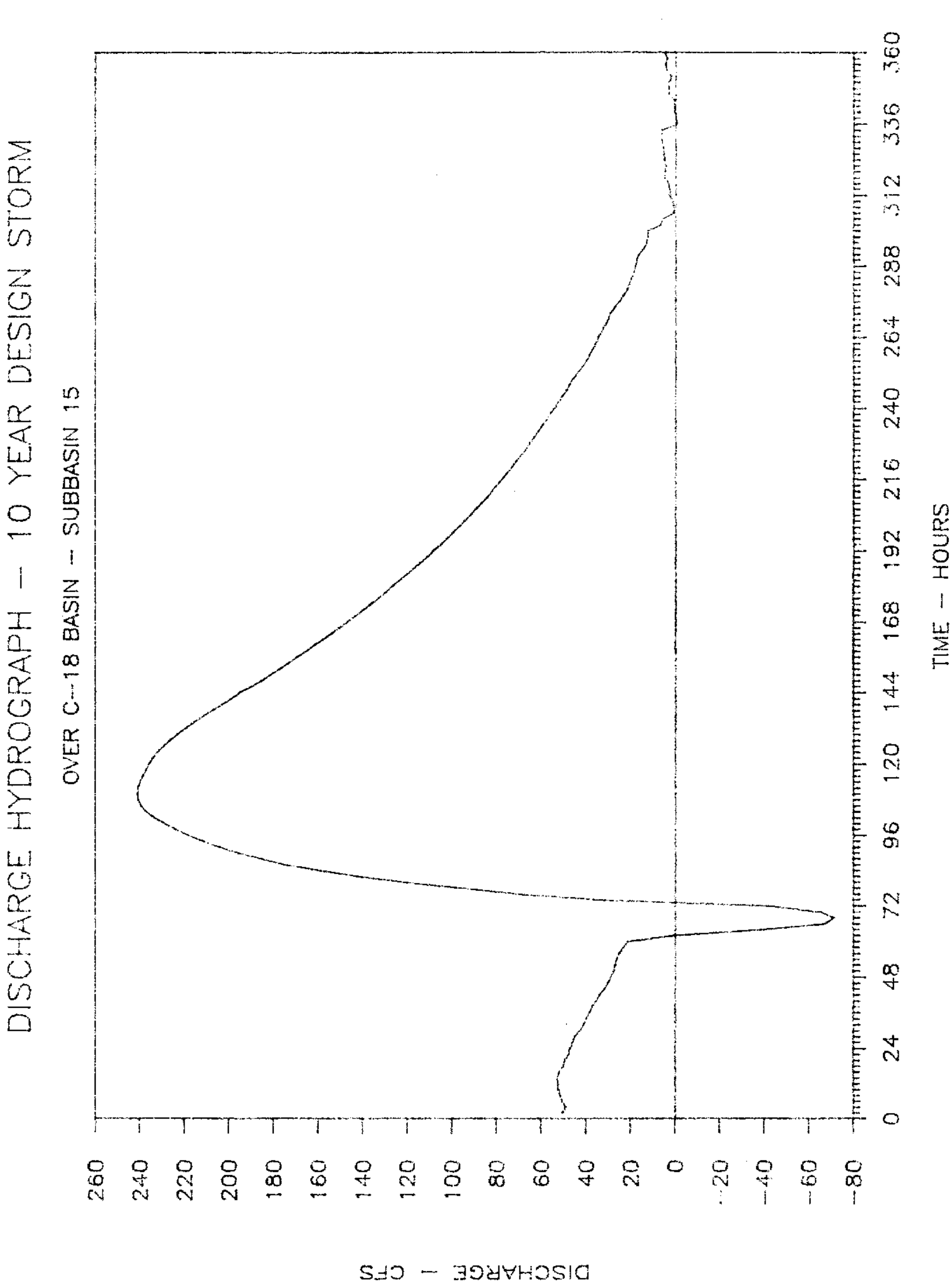


BASIN DISCHARGE HYDROGRAPH — 10 YEAR DESIGN STORM

AT SUBBASIN 13



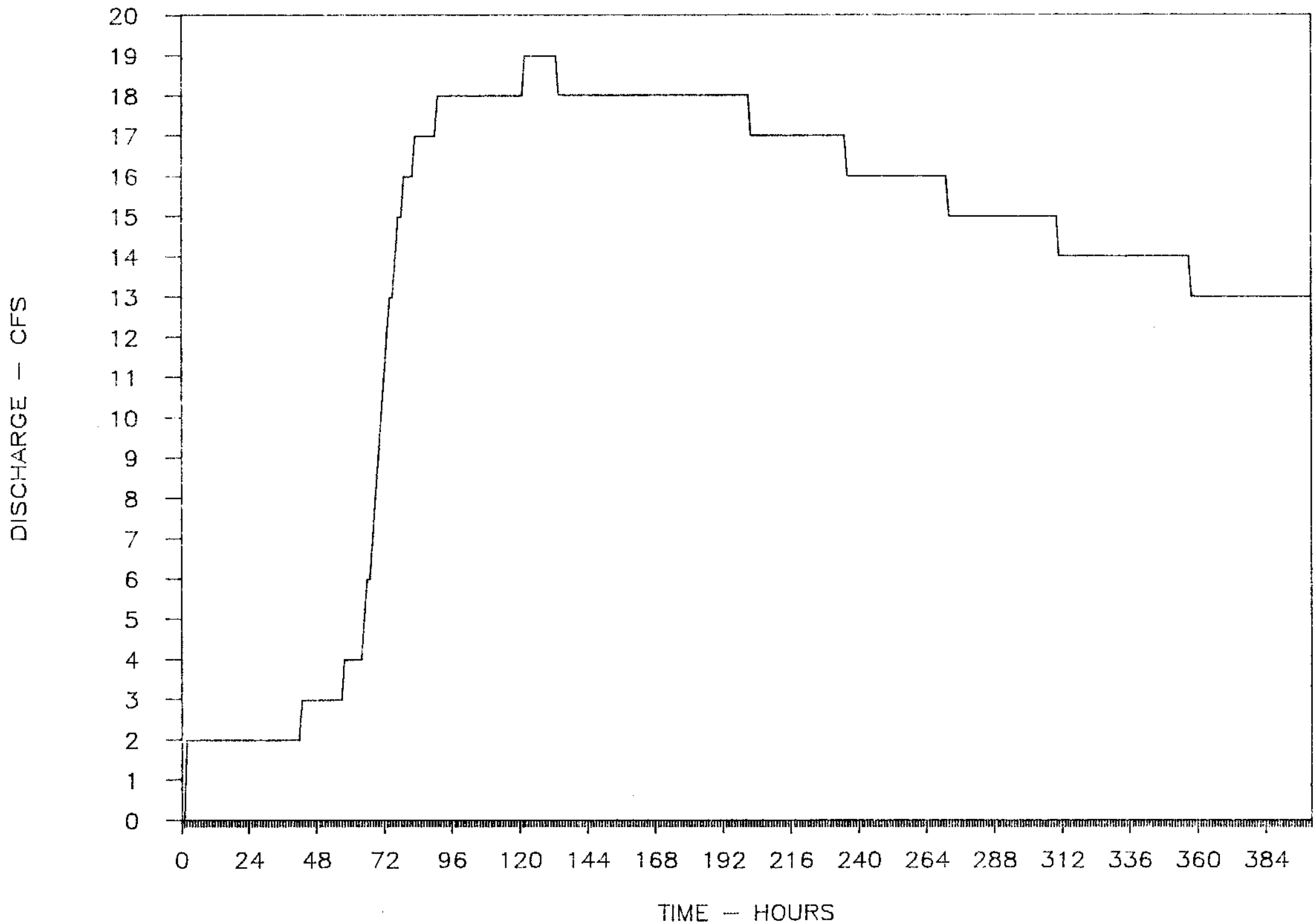

BASIN DISCHARGE HYDROGRAPH — 10 YEAR DESIGN STORM
AT SUBBASIN 14
DISCHARGE — CFS
TIME — HOURS
0
10
20
30
40
50
60
70
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
384

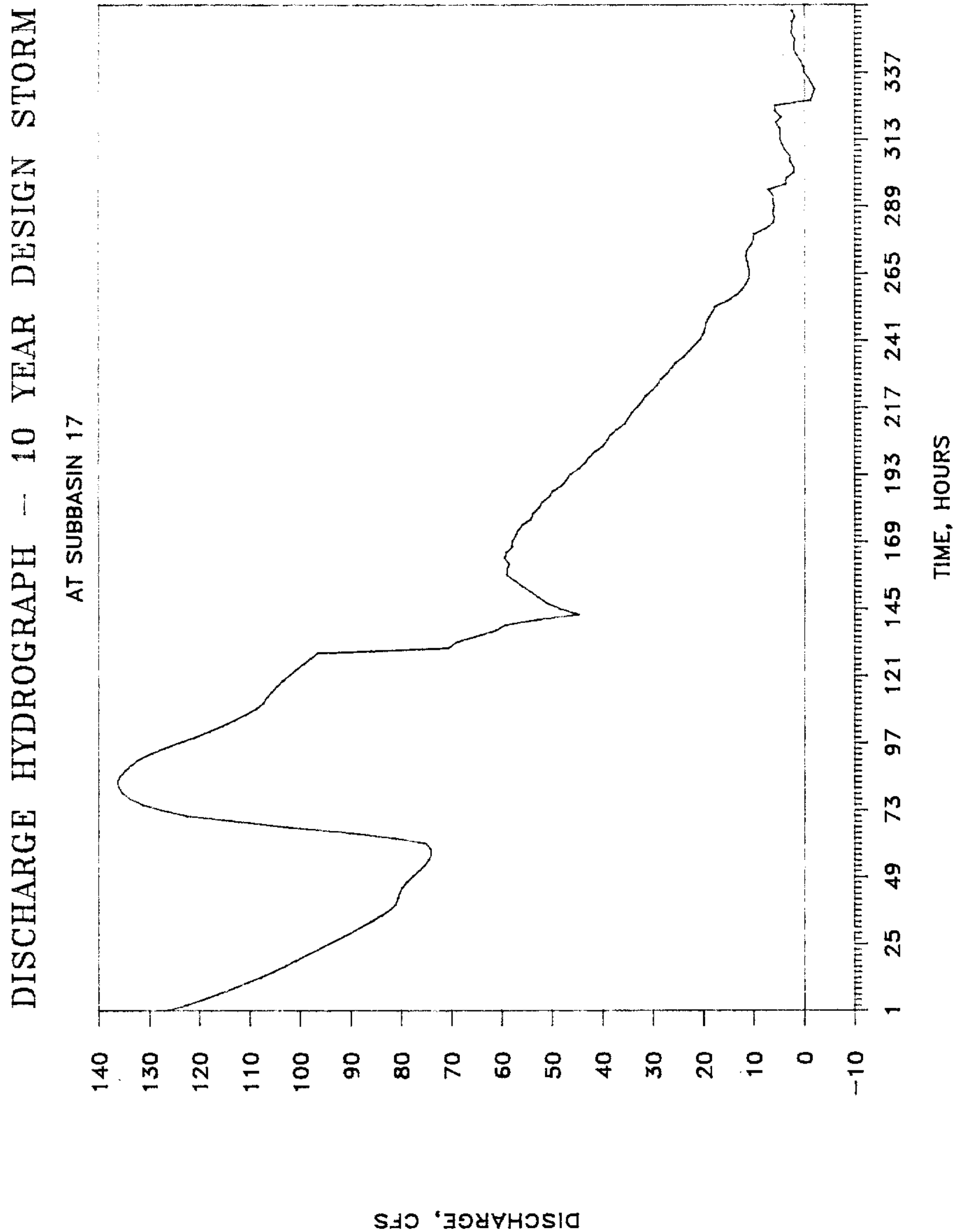


DISCHARGE HYDROGRAPH — 10 YEAR DESIGN STORM
OVER C-18 BASIN — SUBBASIN 15
DISCHARGE — CFS
260
240
220
200
180
160
140
120
100
80
60
40
20
0
-20
-40
-60
-80
TIME — HOURS
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360

BASIN DISCHARGE HYDROGRAPH — 10 YEAR DESIGN STORM

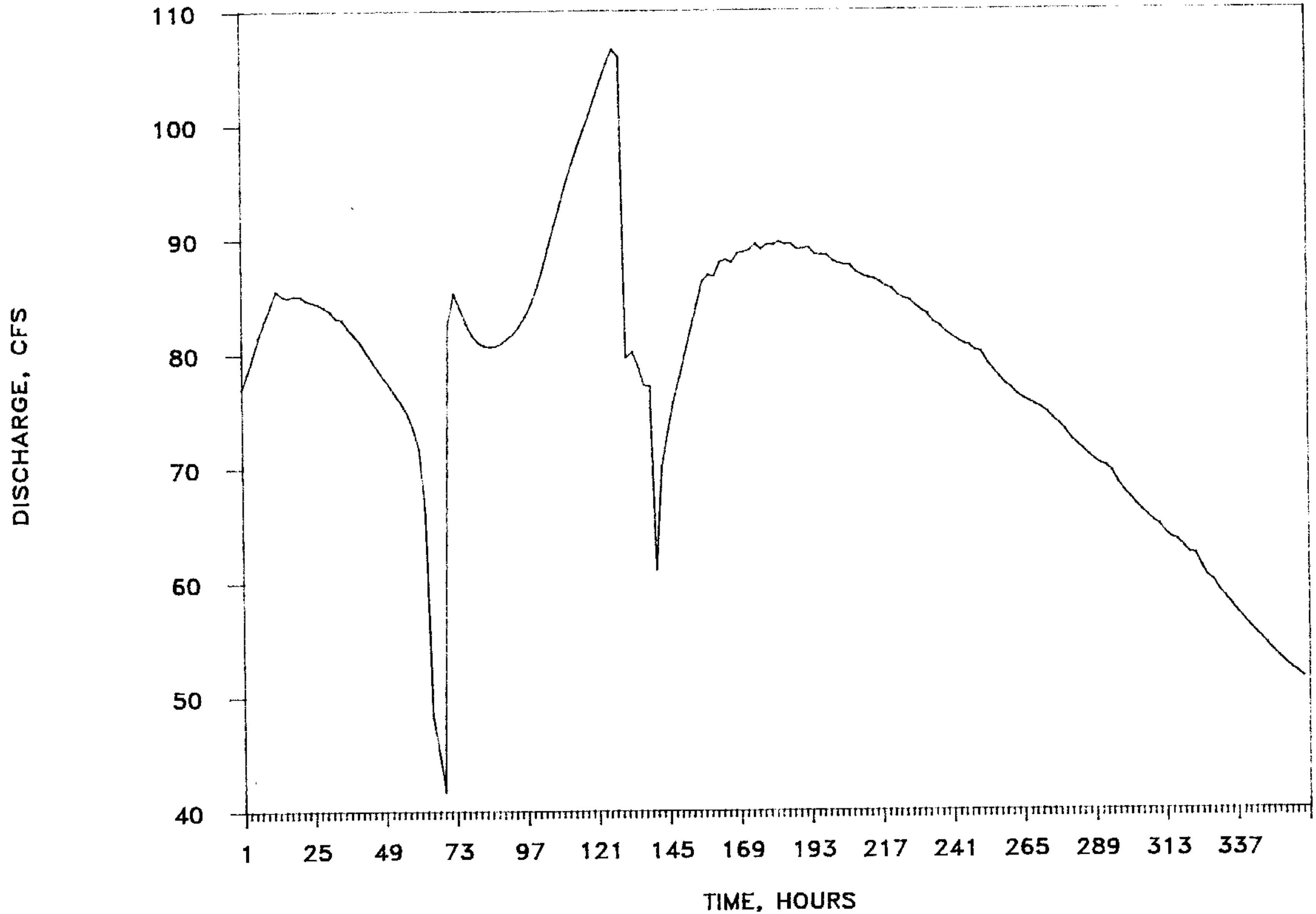
AT SUBBASIN 16

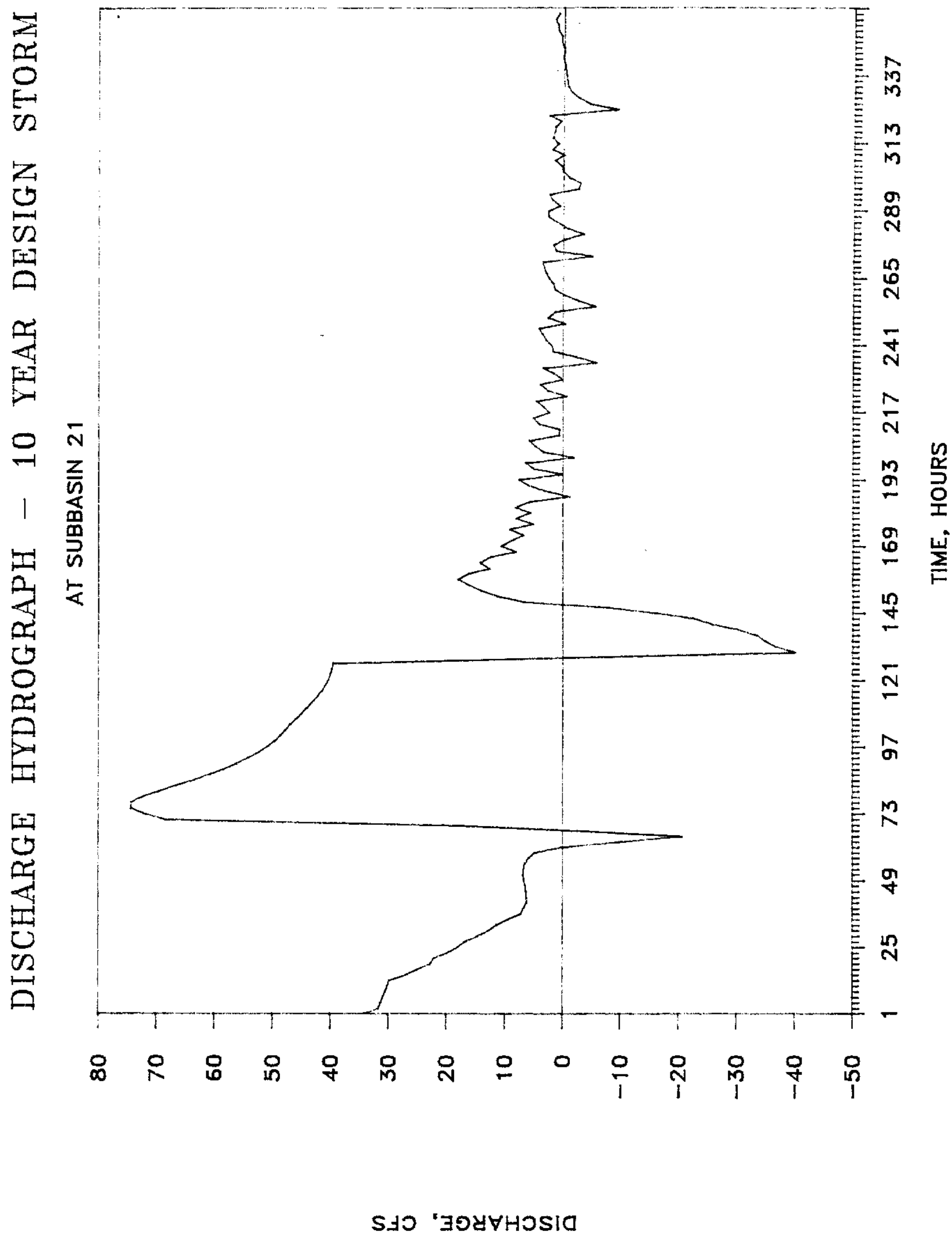



DISCHARGE HYDROGRAPH – 10 YEAR DESIGN STORM
AT SUBBASIN 17
TIME, HOURS
DISCHARGE, CFS
1 25 49 73 97 121 145 169 193 217 241 265 289 313 337
140 130 120 110 100 90 80 70 60 50 40 30 20 10 0 –10

DISCHARGE HYDROGRAPH – 10 YEAR DESIGN STORM

AT SUBBASIN 19




DISCHARGE HYDROGRAPH – 10 YEAR DESIGN STORM
AT SUBBASIN 21
TIME, HOURS
1 25 49 73 97 121 145 169 193 217 241 265 289 313 337
DISCHARGE, CFS
80 70 60 50 40 30 20 10 0 -10 -20 -30 -40 -50

APPENDIX D

STAGE AND STORAGE RELATIONSHIPS FOR EACH SUBBASIN IN THE C-18 BASIN

Subbasin	Elevation Ft NGVD	Storage-AF (Acre-Feet)
3	20	0.00
	21	15.97
	22	583.85
	23	2241.91
	24	4125.47
4 (Reservoir excluded)		
	19	0.00
	20	4.76
	21	70.88
	22	767.00
	23	2235.54
	24	3849.64
	25	5468.10
5	16	0.00
	17	115.91
	18	127.61
	19	144.83
	20	168.52
	21	236.80
	22	761.95
	23	1546.97
	24	2360.24
	24	3175.21
6	17	0.00
	18	1.44
	19	7.18
	20	114.27
	21	673.29
	22	1655.26
	23	2772.69
	24	3911.38
	25	5053.49
7 (Caloosa)		
	16	0.00
	17	42.00
	18	91.00
	19	147.00
	20	213.00
	21	1038.00
	22	2611.00
	23	4184.00
7B (Palm Beach Park of Commerce and offsite parcels)		
	15	0.00
	16	289.89
	17	393.19
	18	504.56
	19	636.36

-continued-

Subbasin	Elevation Ft NGVD	Storage-AF (Acre-Feet)
7B	20	781.77
	21	974.48
	22	1585.64
	23	4149.24
	24	8980.79
	25	15698.02
	26	23177.29
8	16.5	0.00
	17.5	12.00
	18.5	24.00
	19.5	36.00
	20	70.40
	21	273.90
	22	559.50
	23	852.70
	24	1145.95
9	18	0.00
	19	0.11
	20	45.43
	21	390.28
	22	920.91
	23	1454.65
	24	1988.40
	25	2522.15
10	16	0.00
	17	5.54
	18	286.25
	19	1488.88
	20	3802.73
	21	6978.14
	22	10324.09
11	16	0.00
	17	19.58
	18	214.57
	19	533.23
	20	876.84
	21	1221.19
12A	16	0.00
	17	3.60
	18	223.00
	19	1111.70
	20	2245.70
	21	3422.30
	22	4611.10
12B	17	0.00
	18	18.30

-continued-

Subbasin	Elevation Ft NGVD	Storage-AF (Acre-Feet)
12B	19	428.07
	20	2179.90
	21	6089.60
	22	12665.80
	23	20165.19
	24	27736.54
14	15	0.00
	16	62.82
	17	515.98
	18	1248.35
	19	2129.07
	20	3067.20
	21	4008.43
15	15	0.00
	16	21.99
	17	553.52
	18	2457.89
	19	5375.98
	20	8449.00
	21	11567.07
16	14	0.00
	15	12.93
	16	43.04
	17	1196.74
	18	2612.33
	19	4155.83
	20	5699.68
	21	7243.53
17	16	0.00
	17	4.90
	18	199.90
	19	894.89
	20	1941.87
	21	3055.09
18	12	0.00
	13	22.86
	14	244.31
	15	803.94
	16	1506.49
	17	2290.95
	18	3256.42
	19	4597.68
	20	6390.90
	21	8469.08

-continued-

Subbasin	Elevation Ft NGVD	Storage-AF (Acre-Feet)
19	12	0.00
	13	4.11
	14	19.67
	15	45.45
	16	95.21
	17	664.62
	18	2618.65
	19	5403.13
	20	8544.92
20	14	0.00
	15	8.89
	16	193.09
	17	578.82
	18	1042.60
	19	1545.96
	20	2059.86
	21	2576.38
21	15	0.00
	16	6.18
	17	157.68
	18	408.52
	19	666.78
	20	925.05

APPENDIX E

C-51 BASIN PROCEDURE (TRACOR PROCEDURE)

APPENDIX E
C-51 Basin Procedure (Tracor Procedure)

A procedure developed by Tracor, Inc. (1968) was adapted and modified with local data to develop the 30-minute unit hydrograph. This procedure has been applied in water management planning for the western C-51 basin (SFWMD, 1984). The differences between the C-51 basin procedure and the Tracor procedure are:

1. Estimation of peak discharge Q_p
2. Base time t_b

The following paragraphs presents the detail procedures used in the C-51 basin and this study:

1) Time of rise, Tr. This parameter is defined as the time in minutes from the start of direct runoff to the time of peak runoff.

For an "urbanized basin"

$$Tr = 16.44\, \Phi\, L^{0.35}\, S^{-0.049}\, I^{-0.45} \qquad (1)$$

For a "rural basin"

$$Tr = 3.4\, L^{0.223}\, S^{-0.302} \qquad (2)$$

where:

L = length of the main channel (ft)
S = the slope of the main channel (ft/ft)
I = the percent of impervious cover for the subbasin
Φ = an urbanization classification factor with a value of 0.6 to 1.3 (see Table E-1)

2) Peak Discharge, q_p. Runoff rates can be estimated by using the Cypress Creek formula (Stephens and Mills, 1965), which can be expressed as:

$$Q = CM^{5/6} \qquad (3)$$

where:

Q = rate of flow (cfs) in 24 hour period
C = a coefficient based primarily on the level of protection needed
M = drainage area in square miles

$$C = 16.39 + 14.75\,(Pe) \qquad (4)$$

where Pe is the rainfall excess in inches, and can be determined from the basin characteristics by the following equation:

$$Pe = (P-0.2S)^2/(P+0.8S) \qquad (5)$$

P is the 24 hour design rainfall over the basin and S is the potential maximum retention in the basin. Based on the land use and soil types of a subbasin, one can determine a weighted curve number (CN) value associated with an average S value. The maximum discharge developed by Eq. 3 is multiplied by a peak factor, which is obtained from a graph developed by Stephens and Mills, and relates instantaneous peak flow with the size of the drainage area in square miles. After the instantaneous peak was determined, the peak discharge (q_p) of the unit hydrograph was determined, by:

$$q_p = FQ/P_e \qquad (6)$$

where F is the peak discharge factor in the Cypress Creek formula.

3) W_{75}. This is the time in minutes between the points on the hydrograph when the discharge (q_{75}) is 75% of the peak discharge q_p.

$$W_{75} = 1.15 \times 10^4\, A^{0.857}\, q_p^{-0.915} \qquad (7)$$

where A = drainage area in square miles.

4) W_{50}. This is the time in minutes between the points on the hydrograph when the discharge (q_{50}) is 50% of the peak discharge.

$$W_{50} = 2.91 \times 10^4\, A^{0.959}\, q_p^{-0.983} \qquad (8)$$

5) Base Time, T_b. This is defined as the time in minutes from the beginning to the end of surface runoff for a given storm event. The unit hydrograph ordinates prior to the point of inflection can be determined from q_p, Tr, W_{75}, and W_{50}, as presented previously. The remaining portion of the unit hydrograph was calculated based on the method used by the Hydrologic Engineering Center (HEC) of the Corps of Engineers which is an exponential decay function to make sure the total runoff under the unit hydrograph is equal to 1 inch of rainfall excess.

TABLE E-1. URBANIZATION FACTORS USED IN THE TRACOR PROCEDURE

$$\Phi = \Phi_1 + \Phi_2$$

Criterion for Φ_1

0.6	Extensive channel improvement and storm sewer system. Closed conduit channel system.
0.8	Some channel improvement and storm sewers. Mainly cleaning and enlarging existing channel.
1.0	Natural channel conditions.

Criterion for Φ_2

0.0	No channel vegetation.
0.1	Light channel vegetation.
0.2	Moderate channel vegetation.
0.3	Heavy channel vegetation.

APPENDIX F

APPLICATION OF EXTRAN

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APPLICATION OF EXTRAN

In the application of the EXTRAN model the following minor modifications were made:

1. A number of pipes (culverts) were added to the program due to the fact that several pipes (culverts) exist at subbasins instead of using one pipe in the original program for the storm sewer system.

2. Automatic gate operational schemes according to the present setting at S-46 were included in the program.

3. The number of printout steps were increased from 100 to 200 cycles. The total number of junctions and conduits to be plotted was reduced from 20 to 2 to increase computer core storage.

4. The number of storage junctions was increased from 20 to 25 as a maximum.

The use of the storage junction provides a means to handle flow reversals between subbasins and the main C-18 canal due to backwater conditions during severe storm events. The later option was applied to the subbasins where topographic conditions and water control structures were subject to backflow during peak flow hours. This option was not used for those subbasins with elevated stop log spillway risers, such as subbasins 2, 4, 7, 8, 12A, 14, and 16. If the tailwater condition exceeded the top crest elevation of the flashboard of those culverts , the storage option was then applied.

A routing time step of 30 seconds is required to obtain a stable solution. Since there are many short culverts (approximately 40 ft to 100 ft in length) in the system, and a culvert in EXTRAN is considered as a conduit, the stability of the dynamic routing with a 30 second time step was critical. An equivalent longer pipe was developed and used in the model for these culverts (see Appendix A).

At present, the EXTRAN model allows only one inflow to each node (inlet structure to each conduit). Often there is one inflow from each side of C-18 at the same location. The inflow points were separated into two locations in the system to meet current EXTRAN model requirements.

Many short culverts (approximately 40 to 100 ft in length) exist in the system. In order to obtain a stable solution, the following approach was used to extend the culvert length to meet the 30 second routing time step used in the EXTRAN model.

$$(1.49/n_p)\, A_p\, R_p^{2/3} S_p^{1/2} = (1.49/n_e)\, A_e\, R_e^{2/3} S_e^{1/2} \quad \ldots\ldots \quad (1)$$

where:

p = actual pipe,
e = equivalent pipe,
n = Manning's roughness coefficient
A = cross-sectional area
R = hydraulic radius,
S = slope at the hydraulic grade line

Assuming that the equivalent pipe will have the same cross-sectional area and hydraulic radius as the equivalent longer pipe it replaces, then

$$S_p^{1/2}/n_p = S^{1/2}/n_e \quad \ldots\ldots \quad (2)$$

$$\text{since } S = h_L/L \quad \ldots\ldots \quad (3)$$

$$\text{and } L_e = \Delta t\,(gD)^{1/2} \quad \ldots\ldots \quad (4)$$

where

h_L = the total head loss over the conduit length,
L_e = conduit length, ft
Δt = the time step, sec
g = 32.2 ft/sec^2
D = pipe diameter, ft

then

$$n_e = n_p\, L_p^{1/2}/L_e^{1/2}$$